International Journal of Air and Water Pollution

EDITORS

AIR POLLUTION SECTION

R. S. SCORER, *London*
G. NONHEBEL, *London*
C. M. FLETCHER, *London*
W. KLUG, *Darmstadt*
JAMES P. LODGE, *Boulder, Colo.*

WATER POLLUTION SECTION

B. A. SOUTHGATE, *Stevenage,*
W. WESLEY ECKENFELDER, *New York*
E. LECLERC, *Liège*

assisted by an international Editorial Advisory Board

VOL.
5
1961

Volume 5

1961

Copyright © 1963 Pergamon Press Ltd.

PERGAMON PRESS Oxford New York Paris

CONTENTS OF VOLUME 5

NUMBER 1

R. C. Braun and M. J. G. Wilson: The variation of atmospheric sulphur dioxide concentration with altitude *page* 1

A. Turk, J. Morrow, P. F. Levy and P. Weissman: Adsorptive sampling – II. Collection of ethylene on brominated carbon 14

S. Souribajan and J. L. Blumenthal: The application of a copper-silica catalyst for the removal of nitrogen oxides 24

W. L. Dennis: The effect of solid atmospheric pollutants upon the transmission of a collimated beam of infra-red radiation 34

D. W. M. Herbert and J. C. Merkens: The effect of suspended mineral solids on the survival of trout 46

D. W. M. Herbert, J. S. Alabaster, M. C. Dart and R. Lloyd: The effect of china-clay wastes on trout streams 56

Letters to the Editor

W. H. Walton: Aerodynamic capture of particles 75

Abstracts of papers published in *Staub* 76

Air pollution bibliography 80

Reviews 84

VOL.
5
1961

NUMBERS 2/4

R. R. Kountz, J. B. Nesbitt and C. Forney, Jr.: Gas chromatographic studies of sewage metabolisms 85

N. Jaworski, G. W. Lawton and G. A. Rohlich: Aerobic sludge digestion 93

S. Aiba, T. Yamada, A. Yamamoto and S. Shimasaki: Oxygen transfer in the biological treatment of sewage 103

K. H. Mancy and D. A. Okun: Effect of surface active agents on the rate of oxygen transfer 111

J. J. McKeown and D. A. Okun: Effects of surface active agents on oxygen bubble characteristics 113

D. J. O'Connor: Effects of surface active agents on re-aeration 123

A. L. Downing and A. G. Boon: Oxygen transfer in the activated sludge process 131

R. F. Weston and V. T. Stack, Jr.: Fundamentals of operation of entrainment aerators 149

A. A. Kalinske: Power consumption for oxygenation and mixing 157

T. W. Lesperance: The performance and capability of mechanical aerators in activated sludge processing 169

C. Bueltman, G. Wesson, J. Termini and W. Kingsbury: Power requirements for oxygen transfer with turbine aerators 175

P. F. Morgan and J. K. Bewtra: Diffused air oxygen transfer efficiencies 181

A. H. Chasick: Plant tests of Sparger and tube types of diffusers 195

A. W. Busch, N. Myrick, K. W. McKinney and J. W. Peterson: Bench-scale bio-oxidation studies, batch vs continuous systems 199

R. F. Weston and V. T. Stack, Jr.: Prediction of the performance of completely mixed continuous biological systems from batch data 205

W. T. Ingram and G. P. Edwards: The behaviour of filter biota under controlled conditions 215

V. C. Behn: Trickling filter formulations 219

W. E. Howland, F. G. Pohland and D. E. Bloodgood: Kinetics in trickling filters 233

K. L. Schulze: Elements of a trickling filter theory 249

H. E. Orford, H. Heukelekian and E. Isenberg: Effect of sludge loading and dissolved oxygen on the performance of the activated sludge process 251

Brother Joseph McCabe: Mathematical formulation of the biological oxidation process 265

W. W. Eckenfeld, Jr.: Application of kinetics of activated sludge to process design 277

A. Pasveer: Development in activitated sludge treatment in the Netherlands 291
W. von der Emde: Aspects of the high rate activated sludge process 299
R. H. Gould: Realistic application of the newer developments of aeration processes 319
J. Rennerfelt: Bio-oxidation of pulp mill wastes in Sweden 327
E. L. Jones, T. A. Alspaugh and H. B. Stokes: Aerobic treatment of textile mill wastes 331
R. T. Skrinde and G. H. Dunstan: Aerobic treatment of pea processing wastes 339
E. H. Bryan and D. H. Moeller: Aerobic biological oxidation using Dowpac 341
G. W. Reid and J. R. Assenzo: Biological slimes 347
W. J. Oswald: Fundamental factors in stabilization pond design 357
N. L. Nemerow: Accelerated waste water oxidation pond pilot plant studies 357
E. Hurwitz, R. C. Gloppen and J. Roeber: Oxygen absorption capacity of the simplex hi-cone aerator 399
S. H. Jenkins and H. A. Hawkes: Developments in biological filtration in Great Britain 407

AUTHOR INDEX

Aiba, S 187
Alabaster, J. S. 56
Alspaugh, T. A. 415
Assenzo, J. R. 431
Barth, E. F. 161
Behn, V. C. 303
Bewtra, J. K. 265
Bloodgood, D. E. 317
Blumenthal, J. L. 24
Boon, A. G. 215
Braun, R. C. 1
Bryan, E. H. 425
Bueltman, C. 259
Bunch, R. L. 161
Busch, A. W. 283
Chasick, A. H. 279
Dart, M. C. 56
Dennis, W. L. 34
Donovan, E. J. Jr. 125
Downing, A. L. 215
Dunstan, G. H. 423
Eckenfelder, W. W. 361
Edwards, G. P. 299
Emde van der, W. 383
Engelbrecht, R. S. 95
Ettinger, M. B. 161
Forney, C. Jr. 169
Gaudy, A. F. Jr. 95
Gloppen, R. C. 483
Gould, R. H. 403
Hawkes, H. A. 491
Herbert, D. W. M. 46, 56
Heukelekian, H. 335
Howland, W. E. 317
Hurwitz, E. 483
Ingram, W. T. 299
Isenberg, E. 335
Issaac, P. C. G. 145
Jaworski, N. 177
Jenkins, D. 145
Jenkins, S. H. 491
Jones, E. L. 415
Kalinske, A. A. 241
Kingsbury, W. 259
Kountz, R. R. 169
Lawton, G. W. 177
Lesperance, T. W. 253
Levy, P. F. 14
Lloyd, R. 56
Lodge, J. P. (B) 84
Mancy, K. H. 195
McCabe, J. 349
McKeown, J. J. 197
McKinney, K. W. 283
McKinney, R. E. 85, 125
Merken, J. C. 46
Moeller, D. H. 425
Morgan, P. F. 265
Morrow, J. 14
Myrick, N. 283
Nemerow, N. L. 479
Nesbitt, J. B. 169
O'Connor, D. J. 207
Okun, D. A. 195, 197
Orford, H. E. 335
Oswald, W. J. 441
Pasveer, A. 375
Peterson, J. W. 283
Pohland, F. G. 317
Reid, G. W. 431
Rennerfelt, J. 411
Roeber, J. 483
Rohlich, G. A. 177
Schulze, K. L. 333
Shimasaki, S. 187
Skrinde, R. T. 423
Smith, R. M. 125
Sourirajan, S. 24
Stack, V. T. Jr. 233, 289
Stokes, H. B. 415
Symons, J. M. 125
Termini, J. 259
Turk, A. 14
Walton, W. H. 75
Weissman, P. 14
Wesson, G. 259
Weston, R. F. 233, 289
Wilson, M. J. G. 1
Wuhrmann, K. 111
Yamada, T. 187
Yamamoto, A. 187

(B) denotes Book Reviews

REPRINTED OFFSET PHOTOLITHOGRAPHY
BY PAGE BROS. (NORWICH) LTD., MILE CROSS LANE, NORWICH

VOL.
5
1961

Int. J. Air Wat. Poll. Pergamon Press 1961. Vol. 5, No 1, pp. 1–13. Printed in Great Britain.

THE VARIATION OF ATMOSPHERIC SULPHUR DIOXIDE CONCENTRATION WITH ALTITUDE

R. C. BRAUN and M. J. G. WILSON

Imperial College of Science and Technology, London

(*Received* 20 *March* 1961)

Abstract—A light-weight air sampler carried on a small captive kite-balloon has been used to determine atmospheric sulphur dioxide concentrations at heights up to 500 ft. Measurements were made at two sites, one built-up and the other open, in the London area. Concentrations up to 116 p.p.h.m. were observed. On one day the concentration at 500 ft was very nearly the same as that at 100 ft (roof level), whereas on a clear night the concentration at 500 ft was only ⅓ to ¾ of that at 100 ft. On another occasion a high concentration was found at 250–350 ft. Measurements in three fogs showed that the concentration at 500 ft did not differ greatly from that on the ground or at 100 ft.

VOL.
5
1961

1. INTRODUCTION

IT is generally agreed that the concentration of pollution in the atmosphere of cities is greatly affected by variations in the rate at which pollution escapes upwards. Inferences about this upward escape can be drawn from measurements made on the ground, but measurements above ground level might provide more direct means of studying the process. Up to now there have been relatively few measurements of the vertical distribution of air pollutants, and very few measurements in cities. A very brief account of previous work is given below, with particular reference to experimental methods and their limitations for use in cities.

KANNO *et al.* (1959) have drawn conclusions from work at the Keihin Industrial Centre, Tokyo, without giving details of measurements at altitude. GIOVANARDI *et al.* (1959) have determined sulphur dioxide at two stations placed at different heights (109 ft and 342 ft) on a building in a predominantly residential area of Milan. They found that monthly averages of pollution were greater at the lower altitude than at the upper. It is difficult to assess the extent to which convection and other air currents might affect the results of observations made on buildings.

STEWART *et al.* (1958) have studied the vertical concentration profile of a radioactive plume, using a mobile barrage balloon downwind from the chimney. HEWSON and GILL (1943) measured sulphur dioxide concentrations in the Columbia River Valley near Trail B.C., using aeroplanes and a 1200 ft^3 captive kite balloon. The aeroplanes travelled at least one mile while a sample was being taken. The air sampling equipment used with the kite balloon weighed 11 lb and such equipment would be inconvenient for use in cities, where sites for flying large balloons are not readily available. MUNGER (1951) has described the use of a long polyethylene tube attached to a captive balloon for obtaining samples of air from above ground level for sulphur dioxide determinations, but WELCH and TERRY (1960) have recently shown that when air is passed through plastic tubing a considerable proportion of the sulphur dioxide

may be removed. GARTRELL and CARPENTER (1955) have used apparatus carried in a helicopter to measure the distribution of sulphur dioxide from a single source.

The object of the present work was to study conditions near the centre of London, and a balloon seemed the most practical means of lifting the apparatus. Flying the balloon from the ground might have involved risk of entanglement with neighbouring buildings, and the balloon was therefore flown from the roof of one of the highest buildings available near the laboratory. In order to operate from this limited space, and to minimize the cost of the work and the labour required, light-weight apparatus was developed which could be lifted by a balloon of minimum size. Experiments with balloons are usually restricted to periods of low or moderate wind, but these are the conditions in which pollution may be highest.

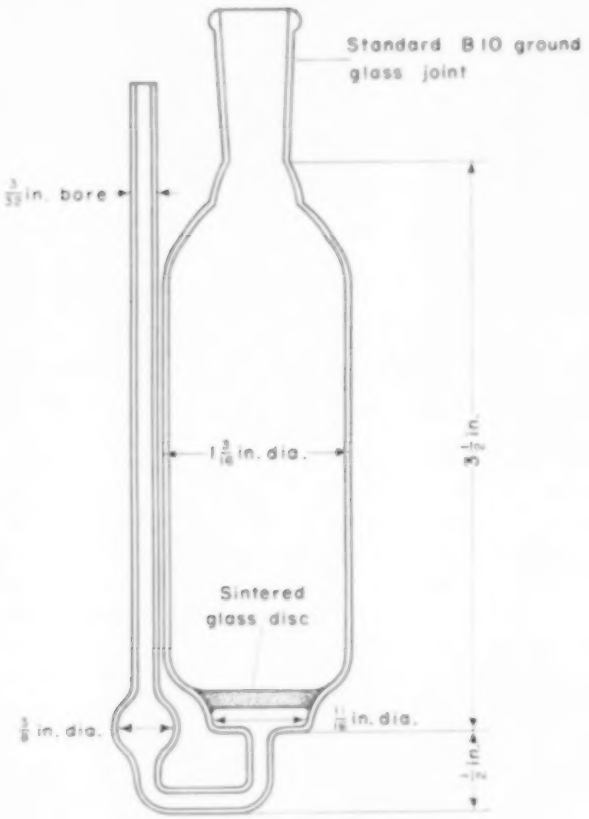


FIG. 1. Diagram of bubbler.

2. EXPERIMENTAL

Air is sucked by a small pump through an absorbing solution in a bubbler. The pump has a rubber diaphragm driven by an A.C. vibrator. The shape of the bubbler (Fig. 1) prevents trapping of bubbles under the sintered glass (No. 0 porosity) and carryover of froth. The bubbler is held in a spring clip and is readily exchangeable.

The scrubbed air is collected in a ½ oz (10 gm) meteorological balloon connected to the outlet of the pump, and the volume of the air is measured on the ground with a gas meter. The sampler weighs 8 oz when ready for use, and it can take air samples at the rate of 0·033 ft³ per min.

Power for the pump is taken from the mains at ground level and supplied to the sampler through a tinsel-on-rayon wire of the type normally used for "deaf aid" cord. The wire weighs about 1 lb per 1000 ft and has a breaking strength of 17 lb. With a resistance of 3·2 Ω per yard and a current drain of 20 mA, the voltage drop across 500 ft of twin wire is about 11 V. A "Variac" transformer is used to compensate for this drop and to adjust the sampling rate. For safety and convenience it is desirable to interpose an isolating transformer and current meter between the mains supply and the sampler.

The kite balloon used to lift the air sampler is made from P.V.C. sheeting, with a capacity of 150 ft³ and a weight of 5 lb 9 oz empty. It is provided with air-filled stabilizer fins and has a total free lift of 4 lb 14 oz at ground level, when filled with hydrogen. This type of balloon has recently become commercially available in Britain. For tethering the balloon a braided nylon cord of 100 lb breaking strength is used. This cord weighs about 1½ lb per 1000 ft and it is marked at 50 ft intervals to indicate the height of the balloon. Separate hand operated winches were used for the tethering cord and the electric supply wire. The cord and wire became entangled when attempts were made to wind both on one winch. The equipment was designed for use up to 1000 ft but a limit of 500 ft was imposed by Ministry of Aviation regulations.

VOL. 5 1961

Sulphur dioxide was determined by the colorimetric method of West and Gaeke (1956). Several investigators (Helwig and Gordon, 1958; McCaldin and Hendrickson, 1959; Welch and Terry, 1960) have studied various aspects of this method, including reproducibility and freedom from bias. It appears that errors due to the analytical procedure are considerably smaller than the effects found in the present work. The method is specific for sulphur dioxide, and sensitive enough to detect 2 p.p.h.m. in a 0·1 ft³ sample of air. The sulphur dioxide is absorbed in a dilute solution of sodium tetrachloromercurate. This reagent protects the dissolved sulphur dioxide from oxidation, and Welch and Terry (1960) have shown that, if necessary, the analysis can be delayed for 72 hr after taking the sample.

Exactly 5 ml of absorbing solution was placed in each bubbler. After the air samples had been drawn through, the bubblers were returned to the laboratory. Exactly 4 ml of solution was withdrawn from each bubbler, mixed with 1 ml of colour reagent, and allowed to stand for 30 min. The optical density was then determined with a spectrophotometer, and the sulphur dioxide content read from a calibration graph prepared using solutions of sodium metabisulphite in absorbing solution.

Using a single sampler, an air sample of 0·1 ft³ could be obtained in 3 min. After taking each sample the balloon had to be wound down to measure the air volume and change the bubbler, so each reading occupied about 15 min. With several samplers at different heights on the balloon cable, connected in parallel and working simultaneously, the output of each pump was reduced and a sampling time of 10 min was used.

3. LOCATION OF EXPERIMENTS

During the winter of 1959–60 sulphur dioxide was determined at heights up to 500 ft by flying the balloon from a roof at Imperial College, Exhibition Road, London,

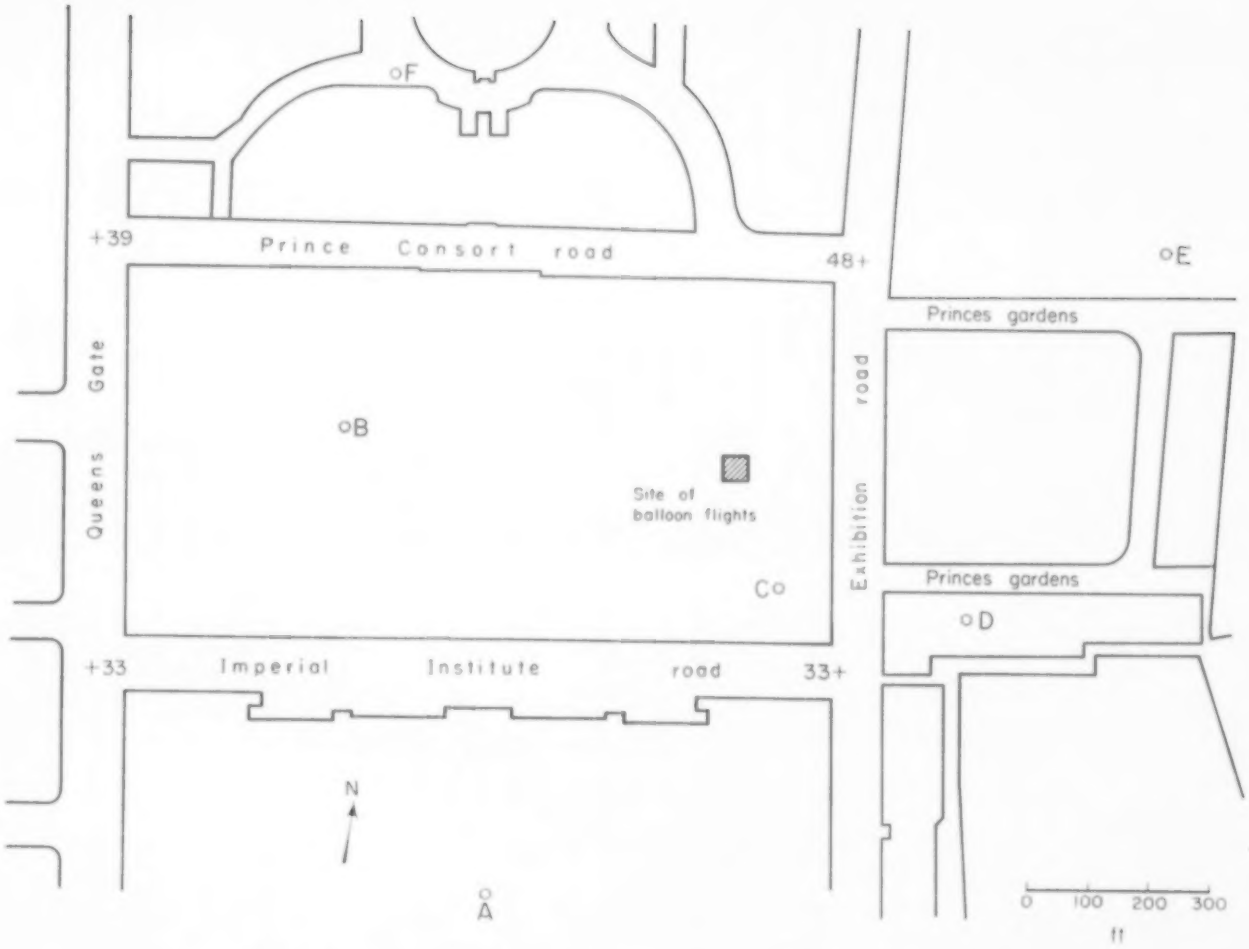


FIG. 2. Area surrounding Imperial College site with location of larger chimneys.

(FIG. 2), in a residential area containing several colleges and museums. The three blocks in the middle of FIG. 2 contain large buildings many of which are about 100–120 ft high. Surrounding blocks contain dwellings 50–60 ft high, and some large buildings. The site is 450 yards from the southern edge of Hyde Park and Kensington Gardens, an open space of about 1 sq. mile in area. The roof is 100 ft above ground and was chosen because it is reasonably accessible and free from obstacles, though very near a large chimney which operated intermittently. No measurements were made while this chimney was operating, and the approximate sulphur dioxide emissions from other large chimneys nearby are given in TABLE 1. The heights of the chimneys are expressed in feet above the level of the ground at Imperial College.

TABLE 1. SO_2 OUTPUT OF LARGER CHIMNEYS NEAR IMPERIAL COLLEGE SITE

Description of Chimney	Height Approx. (ft)	Location (on Fig. 1)	SO_2 emission Approx. (ft^3/hour at N.T.P.)
M.o.W. Boiler Plant	160	A	200
M.o.W. Gen. Station	90	B	440
Unwin Building	70	C	6
47 Princes Gardens	95	D	5
Weeks Hall	130	E	23
Royal Albert Hall	110	F	57

VOL. 5 1961

TABLE 2 gives the distances and bearings from Imperial College of power stations.

TABLE 2

Power Station	Distance (miles)	Bearing
Fulham	2·1	SSW
Lots Road	1·5	S
Acton Lane	4·3	NW
Taylors Lane	4·7	NW
Bankside	3·2	ENE

Both of the experimenters were needed to operate the balloon-borne apparatus, and for this reason measurements were not made at ground level at Imperial College.

A large balloon was occasionally available at Kew Observatory, and the opportunity was taken to make a few measurements there, because the balloon could carry several samplers at once, measurements of lapse rate were being made in the course of other work, and the site differs from that at Imperial College. The Kew site is in an open space about 1 sq. mile in area, and is about 400 yards from the western edge of this space (FIG. 3). The surrounding districts are mainly residential with some industry.

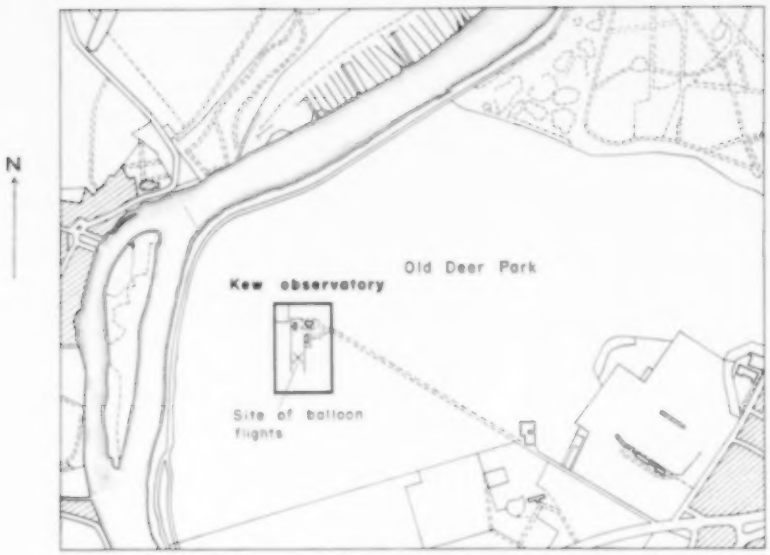


FIG. 3. Area surrounding Kew Observatory site.

4. METEOROLOGICAL DATA

The meteorological data given in this paper are quoted from the records of the Observatory, Kew, the Air Ministry (Kingsway, London), and the Science Museum. From Imperial College these are respectively 6·3 miles WSW, 2·7 miles ENE and 100 yards S. During the present work it was not possible to measure lapse rates at the Imperial College site, but results are quoted from the records made by the Central Electricity Generating Board on the B.B.C. Television Mast at Crystal Palace, $6\frac{3}{4}$ miles SE of Imperial College. These results are given in terms of the height above ground at Crystal Palace; the ground level at Crystal Palace is 320 ft above that at Imperial College.

5. RESULTS

The first measurements were made with a single sampler at intervals of about 15 min, but the sulphur dioxide concentration varied so rapidly with time that vertical differences were partly obscured. A more satisfactory sampling procedure was developed later, using two or more samplers simultaneously at different heights, so that variations with time did not affect the comparison of results at different heights. It was found that the concentration varied with height according to several different profiles, and the various profiles are described below.

5. 1. *Small or zero variation of concentration with height.* Only one sampler was used on the first three occasions described in this section, and the variation of concentration with time must therefore be taken into account. FIG. 4 shows results obtained during a

fog between 15·13 and 16·55 hr on 26 Jan. 60. at Imperial College. The average concentration was greatest at 500 ft and least at 300 ft; the relative differences are smaller than those described in later sections.

Conditions on 26 Jan. 60.

Fog persisted throughout the afternoon, visibility about 300 yards.

Wind—Sci. Mus. and Air Min.—calm; Kew—4–7 knots E.

Lapse rate—Crystal Pal.—15·13 hr stable 0–300 ft, very stable 300–670 ft.

Results obtained at Kew in two other fogs are given in TABLES 3 and 4.

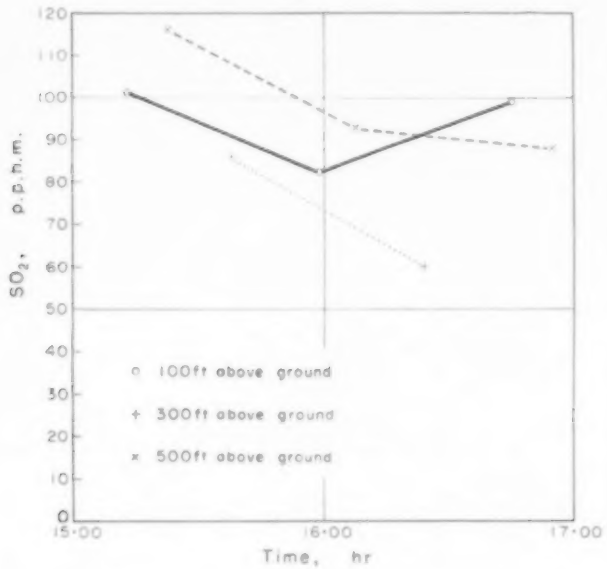


FIG. 4. SO_2 concentrations at heights up to 500 ft during a fog.

TABLE 3. DATE: 1 DEC. 59

Time (hr)	Height above ground (ft)	SO_2 concentration (p.p.h.m.)
12.50	500	33
13.15	300	31
13.40	100	26
13.55	5	23
14.40	500	29

VOL. 5 1961

Conditions.

Dense fog in the early morning, which had thinned a little when readings commenced.

Wind—Kew—2–4 knots ESE; Sci. Mus. and Air Min.—calm.

Lapse rate—Kew—stable from surface to 500 ft.

(Readings taken every 100 ft between 14.29 and 14.43 hr).

TABLE 4. DATE: 7 JAN. 1960

Time (hr)	Height above ground (ft)	SO_2 concentration (p.p.h.m.)
12.00	5	38
12.30	500	56
12.55	100	52
13.10	400	44
13.25	200	45
13.35	300	44
13.40	5	44
17.00	5	34
17.30	500	33
17.45	200	35
18.05	100	44
18.35	400	39

Conditions.

Fog persisted during the day, visibility about 400 yards.

The fog cleared at 17.00 hr, but a shallow layer re-formed near the ground at 18.00 hr.

Wind—Kew—0–2·5 knots SE between 12.00 and 13.40 hr
3 knots E at 17.45 hr
2 knots NE at 18.45 hr
—Sci. Mus. and Air Min.—calm.

Lapse rate—Kew—stable from surface to 500 ft.

(Readings every 100 ft between 16.30 and 16.55 hr).

The results for 1 Dec. 59. might be taken to show some increase with height, but those for 7 Jan. 60. show no marked variation with height.

FIG. 5 shows results obtained by simultaneous sampling on 4 Mar. 60. at Imperial College. Between 16.11 and 17.30 hr the concentrations at roof level differed from those at 500 ft by less than 2·5 p.p.h.m., although the concentration increased from about 20 p.p.h.m. to 30 p.p.h.m.

Conditions on 4 Mar. 60.

Some haze but no definite fog. Some cloud cover at 15.00 hr which had largely disappeared at 18.00 hr.

Wind—Kew—calm up to 16.39 hr, 2 knots ESE at 16.54 hr; 4 knots ESE at 17.56 hr.
—Air Min.—3–7 knots ESE;

Lapse rate—Crystal Pal.—unstable during day, stable during evening (exact times not available).

5. 2. *Decrease of concentration with height.* Readings taken on 4 Mar. 60. after 17.30 hr show the development of a different pattern, the concentrations at 500 ft becoming less than those at 100 ft. The change in pattern began about 15 minutes before the sun set at 17.46 hr.

Fig. 6 shows a larger difference in concentration between the two altitudes. The readings were obtained at Imperial College on 23 Feb. 60. between 21.24 and 22.43 hr and throughout this period the concentration at 500 ft was considerably smaller than at 100 ft.

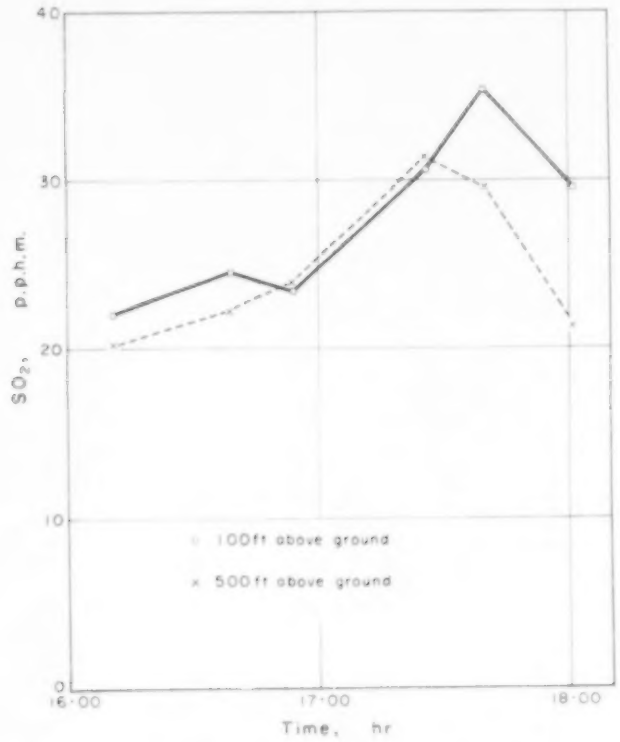


FIG. 5. SO_2 concentrations at 100 ft and 500 ft above ground before and after sunset.

Conditions on 23 Feb. 60.
Clear night with little or no wind at roof level.
A very light NE wind acted on the balloon at 500 ft.
Slight fog commenced at 22.43 hr, and thickened later.
Wind—Air Min., Sci. Mus. and Kew—calm.
Lapse rate—Crystal Pal.—21.24 to 22.43, stable from surface to 600 ft.

5. 3. *Irregular variation of concentration with height.* FIG. 7 shows results obtained between 15.25 and 16.25 hr on 5 Feb. 60 at Kew. There had been fog in the morning,

VOL. 5 1961

but this had cleared when readings commenced. The sulphur dioxide concentrations were low and variable both at ground level and at 500 ft.

Conditions on 5 Feb. 60.

Rain fell between 16.00 and 16.25 hr.

Wind—Kew—calm until 15.43 hr
7 knots N at 16.02 hr
4 knots N at 16.25 hr.

Lapse rate—Crystal Pal.—15.25 hr unstable 0–400 ft.

Kew—not measured.

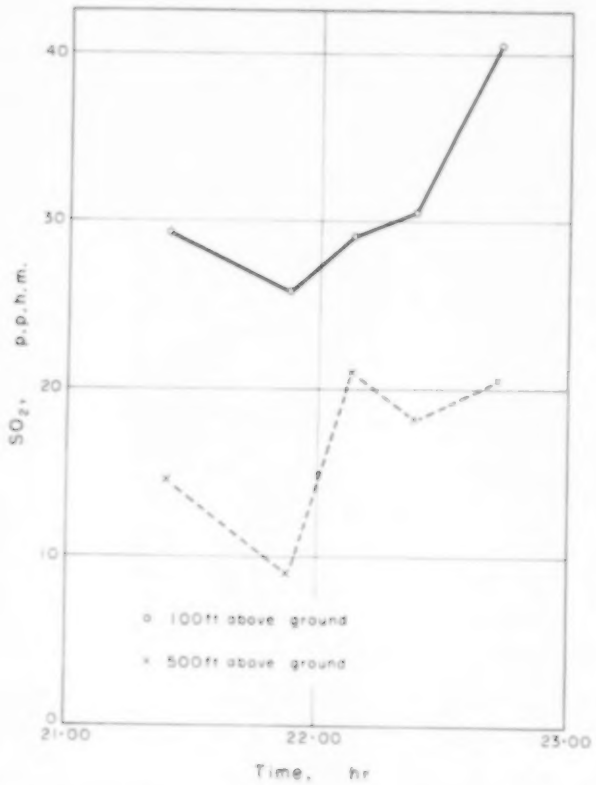


FIG. 6. SO_2 concentrations at 100 ft and 500 ft above ground on a calm clear night.

5. 4. *High concentration between 250 and 350 ft.* TABLE 5 shows results obtained at Kew using five samplers at 0, 50, 150, 250 and 350 ft on the balloon cable. Four separate surveys were made between 16.30 and 22.13 hr on 25 Mar. 60. On three occasions the concentration was much greater between 250 and 350 ft than at lower levels. The source (or sources) of this high concentration could not be identified with certainty; possible sources were a gasworks 1½ miles away and/or a group of power stations 5–7 miles away.

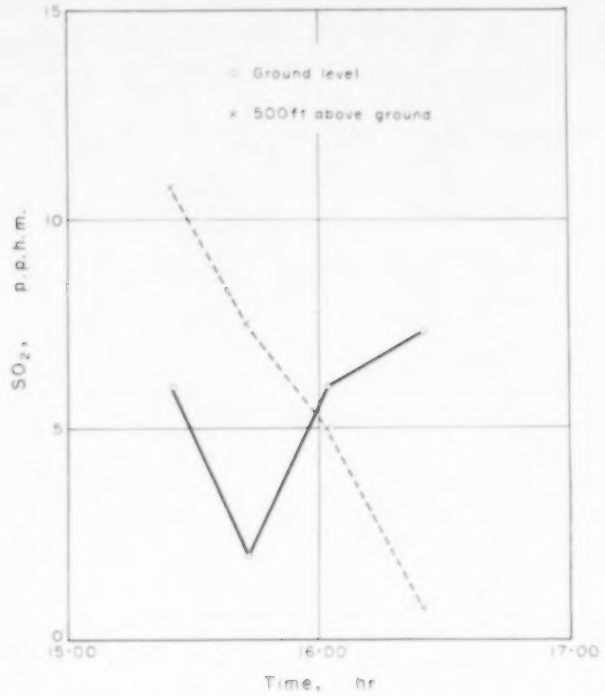


FIG. 7. SO_2 concentrations at ground level and 500 ft above ground on a day of variable weather.

TABLE 5. DATE: 25 MAR. 60

Time (hr)	Height (ft)	SO_2 concentration (p.p.h.m.)
16.35	250	28
,,	150	6
,,	0	8
18.07	350	15
,,	250	12
,,	150	10
,,	50	12
,,	0	13
21.15	350	48
,,	150	11
,,	50	7
,,	0	9
22.08	350	38
,,	250	15
,,	150	13
,,	50	8
,,	0	23

VOL. 5 1961

Conditions

Cloudy during the afternoon, clearing during the evening with clear sky after 21.00 hr.

Wind—Kew—Time (hr)	Speed (Knots)	Direction (degrees)
16.30	9	50
18.03	12	20
21.10	12	20
22.03	10	20

Lapse rate—Kew—neutral from surface to 350 ft (readings at 5, 200, 300 and 350 ft between 18.50 and 19.05 hr).

6. DISCUSSION

The present work shows that pollution may be distributed vertically in various patterns, or profiles, but the results are not complete enough to justify a detailed quantitative treatment, and only a few points will be mentioned.

The concentration profile of pollution above a city must depend on several factors, including the stability of the atmosphere, the wind velocity gradient, and the height at which the pollution is emitted. The profile at a particular time and place will depend not merely on the local conditions at the time but also on the previous history of the air in its passage from the edge of the city to the sampling point. This previous history cannot, in general, be determined completely from observations at a single point.

The effect of wind velocity gradient is clear from the consideration that pollution is diluted by a flow of air which increases with height. Thus a large wind velocity gradient tends to produce a decrease of concentration with height. When the wind velocity gradient is large, the velocity at 500 ft above ground might well be about twice that at 100 ft above ground at the Imperial College site. A decrease of concentration with height is also to be expected when the atmosphere is stable and the upward movement of pollution is hindered. High values of wind velocity gradient are often associated with atmospheric stability, and it is therefore difficult to separate the effects of these two factors on the profile of pollution, though their combined effect has been observed. Thus the decrease with altitude found on a clear night (FIG. 6) is consistent with the fact that the atmosphere was stable, and the uniform concentration found on a clear day (earlier part of FIG. 5) is consistent with instability. Also, the change of pattern shown in FIG. 5 occurred at a time when the atmosphere was changing from unstable to stable.

Comparison of the results of this work with theory appears to present a difficulty, since the diluting effect due to increase of wind velocity with altitude, which could account for a significant part of the effect shown in FIG. 6, is not usually allowed for explicitly in theoretical treatments of atmospheric diffusion.

The restriction of the balloon to a maximum height of 500 ft has been a serious limitation in the present work. To reach the sampling point at Imperial College, air must have travelled a distance of 50,000–100,000 ft from the periphery of the city. In comparison, 500 ft is a small height, and there are reasons for believing that much of the pollution produced by the city must have passed above the balloon. One reason is that the effective plume height from some power stations may be at least 500 ft. Another is that according to LUCAS (1958) pollution may spread upwards at an angle

of about 1·5° even in stable conditions. Thus some of the pollution emitted near the ground would reach a height of about 2600 ft in travelling a horizontal distance of 100,000 ft. Pollution from sources nearer the sampling point would, of course, be found at a lower altitude.

A city smaller than London might therefore be more suitable for future aerial surveys of pollution, especially if there were few very high chimneys and the maximum height of the balloon were not too severely restricted by the presence of aircraft. It might then be possible to reach up to nearly pure air, and so draw up a complete balance between the amounts of pollution liberated near the ground and the amounts found at various heights above the city.

7. CONCLUSIONS

The equipment described in this paper is simple, reliable, readily transported and light enough to be lifted by a relatively small balloon. It has been used to determine sulphur dioxide in the atmosphere up to a height of 500 ft.

Several different patterns have been recognized for the variation of sulphur dioxide concentration with height. The concentration varied comparatively little with height in three fogs and on one clear day. A marked decrease with height was found on one clear night. In three experiments, by day and by night, the concentration was much higher at 250–350 ft than at lower levels. Rapid and irregular variations have also been detected.

Acknowledgements—We are indebted to Dr. K. H. Stewart for making available the facilities at Kew, and to the Central Electricity Generating Board for the loan of records of lapse rate made at Crystal Palace. Other meteorological data are reproduced by permission of the Meteorological Office and the Science Museum. R. C. Braun is indebted to the authorities of Hendon Technical College for leave of absence to carry out this work.

VOL.
5
1961

REFERENCES

Gartrell F. E. and Carpenter S. B. (1955) Aerial sampling by helicopter: a method for study of diffusion patterns. *J. Met.* **12,** 215–219.

Giovanardi A., Grosse E. and de Fraja Frangipane E. (1959) Researches on the Atmospheric Pollution of the City of Milan. *Int. Clean Air Conf. Lond.* Paper **IV,** 38.

Helwig H. L. and Gordon C. L. (1958) Colorimetric method for continuous recording analysis of atmospheric sulphur dioxide. *Anal. Chem.* **30,** 1810–1814.

Hewson W. E. and Gill G. C. (1943) Meteorological Investigations in Columbia River Valley. *U.S. Bur. Mines Bull.* 453, Part II.

Kanno S., Fukui S., Ikeda H. and Ono Y. (1959) Atmospheric Sulphur Dioxide Concentrations Observed in the Keihin Industrial Centre and Their Relation to Meteorological Elements. *Int. J. Air. Poll.* **I,** 234–240.

Lucas D. H. (1958) The Atmospheric Pollution of Cities. *Int. J. Air Poll.* **I,** 71–86.

McCaldin R. O. and Hendrickson E. R. (1959) The Use of a Gas Chamber for Testing Air Samplers. *Amer. Industr. Hyg. Assoc. J.* **20,** 509–513.

Munger H. P. (1951) Techniques for study of Air Pollution at Low Altitudes. *Chem. Eng. Progr.* **47,** 436–439.

Stewart N. G., Gale H. J. and Crooks R. N. (1958) The Atmospheric Diffusion of Gases Discharged from the Chimney of the Harwell Reactor BEPO. *Int. J. Air Poll.* **I,** 87–102.

Welch A. F. and Terry J. P. (1960) Developments in the Measurement of Atmospheric Sulphur Dioxide. *Amer. Industr. Hyg. Assoc. J.* **21,** 316–321.

West P. W. and Gaeke G. C. (1956) Fixation of Sulphur Dioxide as Disulphitomercurate and subsequent Colorimetric Estimation. *Anal. Chem.* **28,** 1816–1819.

Int. J. Air Wat. Poll. Pergamon Press 1961. Vol. 5, No. 1, pp. 14–23. Printed in Great Britain.

ADSORPTIVE SAMPLING II. COLLECTION OF ETHYLENE ON BROMINATED CARBON*

AMOS TURK, JACK MORROW, PAUL F. LEVY and PAUL WEISSMAN

Department of Chemistry, The City College of New York

Abstract—A system for ambient-temperature sampling of atmospheric organic gases and vapors is proposed which comprises two stages: (a) activated carbon and (b) brominated activated carbon. Material which is adsorbed and retained in the first stage may be recovered for analysis. Material not so retained passes through to the second stage, where it is subject to bromination and retention as the brominated product.

It is shown that ethylene in a dilute ethylene-air mixture will be efficiently retained by a thin bed of brominated activated carbon, with conversion of the ethylene to 1, 2-dibromoethane. Experiment shows that only a portion of the adsorbed bromine is available for rapid reaction with the ethylene, the remaining bromine reacting more slowly. If bromine is used in theoretical excess of the ethylene to be sampled, the procedure lends itself to convenient outdoor operation.

The adsorbed brominated product may be recovered by desorption into evacuated cold traps. Laboratory procedures are given. The product is analyzed by infrared spectrometry or gas chromatography.

An application example in which ethylene is identified in outdoor air with adsorptive sampling under ambient conditions is described.

INTRODUCTION

ADSORPTIVE sampling of outdoor atmospheres for non-selective collection of organic vapors is best effected by the use of activated carbon (TURK, SLEIK and MESSER, 1952). At ambient temperatures, activated carbon without chemical impregnation is suitable for the retention only of vapors whose molecular weights are significantly higher than that of air. Smaller molecules may be retained by refrigerated carbon (SMITH, 1959; WEST, *et al.*, 1958), but requirements of cooling during sampling and transport are inconvenient. During refrigeration below 0° C, clogging of the carbon with ice may invalidate the sampling.

Ethylene is an important example of a light gas (molecular weight 28) whose retention by physical adsorption alone is inadequate for atmospheric sampling. Sampling with bromination, however, increases the molecular weight to 188, thereby producing a product which is retained tenaciously by physical adsorption on activated carbon.

THE USE OF BROMINATED CARBON

Activated carbon can adsorb and retain large quantities of bromine, which in the sorbed state is chemically available as a brominating agent (RAY, 1955; WIRTH,

*A portion of the work herein described was presented at the 136th Meeting of the American Chemical Society, Atlantic City, N.J., September 1959.

1953). Brominated carbon is stable at ambient temperatures, and all brominated organic compounds are strongly retained on activated carbon (TURK, 1960). If brominated carbon were to be considered as a sampling medium, however, the collected material would not be the pollutant gases and vapors themselves, but their brominated products, which would then have to be related to their unbrominated atmospheric precursors. To minimize the analytical complications involved in

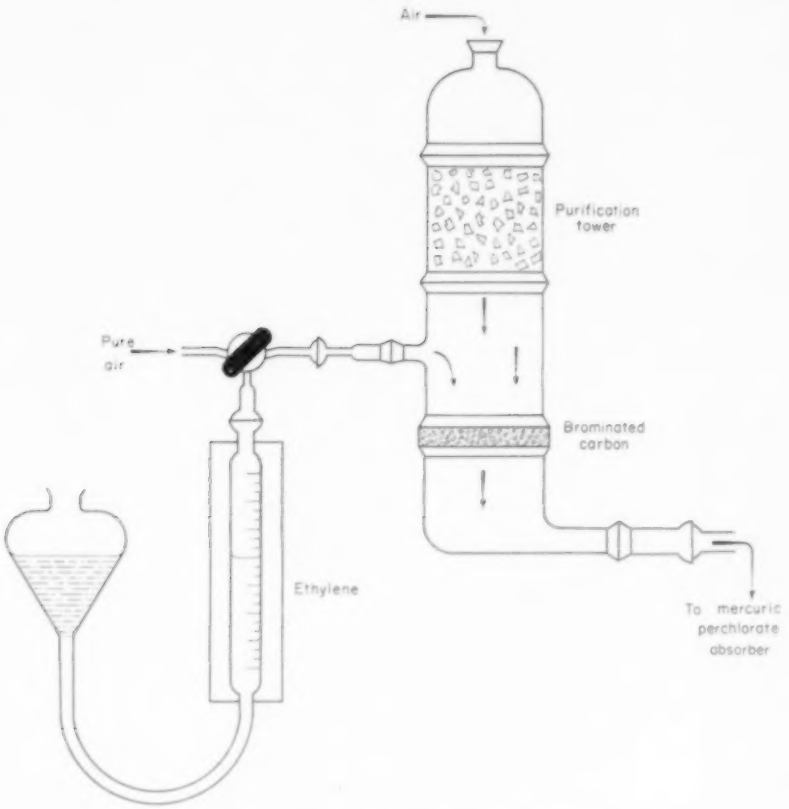


FIG. 1. Brominated carbon absorber for ethylene.

brominating a multi-component vapor mixture, a system was conceived which entailed passage of air to be analyzed through successive beds of (a) activated carbon and (b) brominated activated carbon, both at ambient temperature. The bromination reaction then involves only those relatively few components which pass through the ambient-temperature bed of activated carbon. Among hydrocarbons, such a two-stage activated carbon-brominated carbon system will allow only the lightest paraffins (e.g., methane) to pass through without efficient retention. In addition, the first (unbrominated) stage provides for routine adsorptive sampling of heavier vapors.

VOL. 5 1961

ADSORPTION OF ETHYLENE

Ethylene, being the lightest olefin, was chosen as the test gas to evaluate the performance of the activated carbon-brominated carbon system. Apparatus (FIG. 1) was set up to permit the admixture of a measured volume of ethylene with dry purified air, and to stream the combined gases first through a thin bed of brominated carbon, and then through a chemical absorbing solution to retain any non-adsorbed ethylene. The system was constructed of 3 in. and 1 in. borosilicate glass piping, sections of which were joined by Teflon gaskets and held together with metallic flange sets. The 1 in. sections were sealed by means of glass ball and socket joints to other parts of the system. Purification of air was effected by passing it through successive layers of glass wool, silica gel, brominated carbon, activated carbon, and glass wool. The absorbing solution used was mercuric perchlorate in aqueous perchloric acid (GROSS). The brominated carbon bed was retained in a $\frac{3}{4}$ in. borosilicate glass spacer, held with Teflon gaskets between glass piping sections. The carbon was mechanically supported by a layer of perforated stainless steel sheet (37 per cent open area).

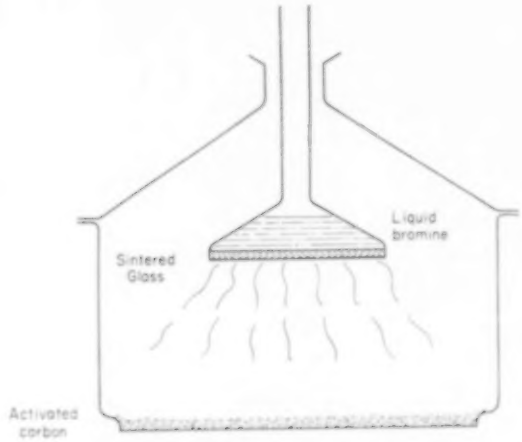


FIG. 2. Vapor Phase bromination of carbon.

BROMINATION PROCEDURE

The impregnation of the activated carbon with bromine was carried out by one of the following methods:

(a) The required amount of pure liquid bromine was poured into granular (6–14 mesh) activated coconut shell or bituminous carbon in a glass flask, stoppered, shaken, and allowed to progress toward equilibration for at least one day;

(b) The activated carbon was spread in a large desiccator jar to a layer which was two or three granules thick. Bromine vapor was introduced slowly by allowing it to leak through a sintered glass filter (FIG. 2). Evaporation at the bottom of the filter was rapid enough to prevent any liquid bromine from dripping onto the carbon. Instead, only vapor descended and came in contact with the thin carbon bed.

Samples of carbon impregnated with 5, 10 and 15 per cent of their weight of bromine were prepared.

The uniformity of bromination was checked by setting randomly selected individual granules from a batch of brominated carbon onto porcelain spot plate depressions. Each granule was then covered with starch-iodide solution, appearance of color being a test for free bromine available for reaction. Brominated carbon prepared from liquid bromine (method (a)), and allowed to stand for one month, showed a strong positive bromine indication in about one granule in six, the remaining granules giving no visible reaction. Brominated carbon prepared in vapor phase (method (b)) showed a relatively uniform bromine indication in 19 out of 20 granules.

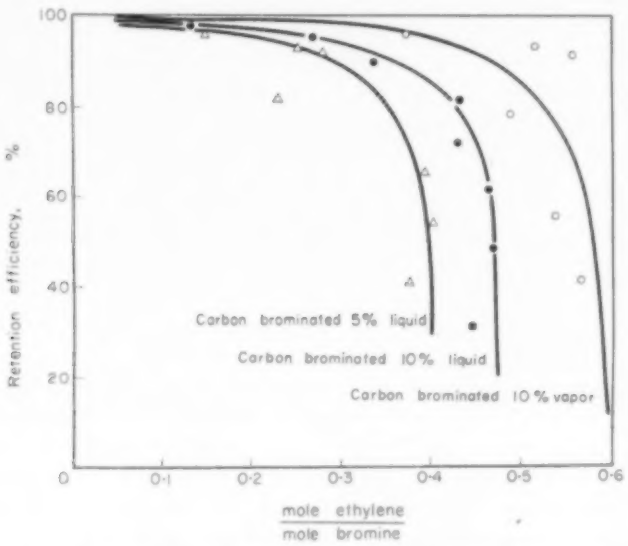


FIG. 3. Retention efficiency and capacity of brominated carbon for ethylene.

EFFICIENCY AND CAPACITY OF ETHYLENE RETENTION BY BROMINATED CARBON

For a determination of retention efficiency, a measured quantity of ethylene gas was introduced into the pure, dry air stream which flowed through the $\frac{3}{4}$ in. (1·9 cm) brominated carbon bed at a rate of 2·5 liters per min. The total bed volume was 91 cm^3 and the nominal average duration of contact between bed and air stream was therefore 2·2 seconds. The fraction of the carbon bed which is gross void space (as measured by water displacement after the carbon is saturated with CCl_4 vapor at 0° C) is 38 per cent; actual duration of contact is therefore 0·8 seconds. A volume of 50 ml of ethylene was generally introduced over a period of about 20 min; ethylene concentration in the air stream was therefore of the order of 0·1 per cent. All residual ethylene in the side tube was flushed into the main stream with dry, purified air.

The air stream, after traversing the carbon bed, entered the mercuric perchlorate absorber for ethylene. For analysis, the resulting complex was oxidized at 50–55° C. for 45 min with a perchlorato-cerate solution. The oxidized mixture was then diluted and titrated with 0·1 N sodium oxalate solution, using nitroferroin as an indicator.

VOL. 5 1961

From the difference between this titration, and the one of a parallel blank run containing no ethylene, the volume of ethylene absorbed in the mercuric perchlorate solution is calculated. Since this represents ethylene *not retained* by the brominated carbon, the retention efficiency is:

$$\text{Retention Efficiency (\%)} = \frac{\text{Volume added} - \text{Volume recovered}}{\text{Volume added}} \times (100)$$

With experiments in which the adsorbent bed was omitted, it was found that 90 per cent of the measured ethylene introduced into the system could be analytically accounted for. This recovery was independent of the amount of ethylene analyzed.

FIG. 3 gives information on ethylene retention efficiencies and capacities. Retention efficiency (ordinate) is plotted against the ratio of moles of ethylene added to the air stream/moles of bromine added to the carbon.

It is seen that retention efficiency is very high over a range of capacity sufficiently broad to indicate that the system may be valid for sampling of outdoor atmospheres.

From the 5 per cent brominated carbon, retention efficiency remains high (over 90 per cent) until the molar ethylene/bromine ratio reaches a value of about 0·3. Carbon brominated to the extent of 5 per cent or 10 per cent gives no bromine odor even on close sniffing, and no irritating action to the skin in routine handling. In view of the demonstrated heterogeneity of bromine distribution, it is noteworthy that this brominated carbon acts so efficiently over almost a third of its theoretical range of capacity. Unbrominated carbon, in parallel experiments (curves not shown) will adsorb a quantity of ethylene, but so much of it is desorbed on continued passage of pure air, that the system is analytically useless. It is postulated that adsorption of ethylene may occur on brominated or non-brominated sites but that desorption occurs only from non-brominated sites. One may ask why so much of the bromine is unavailable for reaction under the conditions of the experiment. This circumstance may be related to the distribution of the bromine in thick layers (capillary condensation), with reaction occurring at the surface and rendering the deeper liquid temporarily unavailable. Alternatively, bromine may become unavailable by virtue of its absorption between graphitic layers of the carbon. Either concept is consistent with the following evidence:

(1) For the same degree of bromination, more bromine is available for reaction from a more uniformly brominated carbon (FIG. 3). The more uniformly brominated sample may be presumed to have less deeply-layered and hence less unavailable, bromine.

(2) When a bed of brominated carbon becomes "exhausted" (inefficiently retentive) from passage of ethylene through it, it will regenerate, or recover some of its efficiency, on standing. This recovery of efficiency is more marked for uniformly than for heterogeneously brominated carbon. This effect may be related to the slow diffusion of bromine from deep capillaries or from graphitic layer interstices, a process which would be expected to be more effective for samples of more uniform bromination.

(3) After a sample of brominated carbon has become "exhausted" by passage of ethylene through it, additional bromine may be removed from the carbon by heating it under evacuation to about 250° C and trapping the distillate at dry-ice

temperature. A quantity of free bromine is thereby obtained. Such recovery of bromine from "exhausted" brominated carbon is not effective at 100° C and certainly not at ambient temperatures.

IDENTIFICATION OF ETHYLENE FROM ITS BROMINATED PRODUCT

Addition of bromine to ethylene produces 1,2-dibromoethane. If identification of this product is to be used as analytical indication of atmospheric ethylene, it is important to verify that the product is retained on the brominated carbon during sampling and that it can later be recovered, and to determine the extent to which abnormal reaction products may be produced.

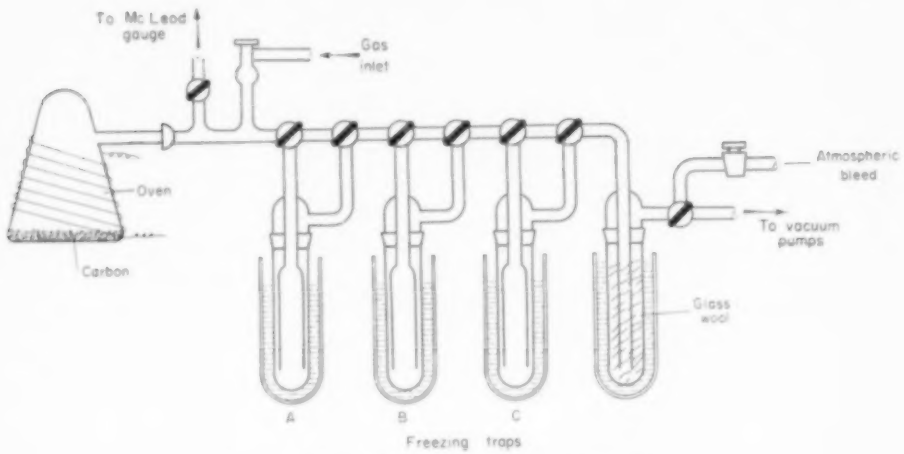


FIG. 4. System for desorbing brominated carbon.

To verify the retention of dibromoethane, a sample of brominated carbon was allowed to react exhaustively with ethylene. Air was then passed through this sample for one-half hour, the effluent stream being continuously bubbled through an absorber containing saturated alcoholic silver nitrate solution. The solution, on subsequent heating, showed no turbidity. As a control, trace amounts of dibromoethane vapor, introduced into the silver nitrate solution, gave a distinct white precipitate on heating.

The recovery of the adsorbate, from either the unbrominated or the brominated stage, is effected by desorption into evacuated cold traps. Analysis of the trap contents confirmed the presence of 1,2-dibromoethane. The apparatus, procedure, and results of individual experiments merit more detailed description:

DESORPTION PROCEDURE

Referring to FIG. 4, the carbon to be stripped is placed in the "oven" (not yet connected to the vacuum train), a modified thick-walled flat-bottomed flask wrapped with heating wire and insulated. The carbon bed, being distributed over the wide area of the oven bottom, is not more than 5 mm in depth, thus offering a short mean path for desorbed vapor molecules to leave the carbon granules. The system, exclusive of the

VOL. 5 1961

oven, is evacuated to about 10^{-5} mm Hg, during which process its various components are heated with electrical tape to promote degassing. The traps are then cooled by immersing them in liquid nitrogen, the oven containing the carbon to be desorbed is attached, and the stopcocks are set so that the vapor must pass through all the traps in series. Oven heat is then applied and the desorption run is carried out.

It was found that practically all the desorbed material appeared in the first trap. Since the stopcock system can by-pass any combination of traps *A*, *B* and *C*, the traps may be conveniently used sequentially to permit appraisals of step-wise increases in oven temperatures, or increased durations of desorption. The last trap (*D*) is always kept in-line to protect the pumps.

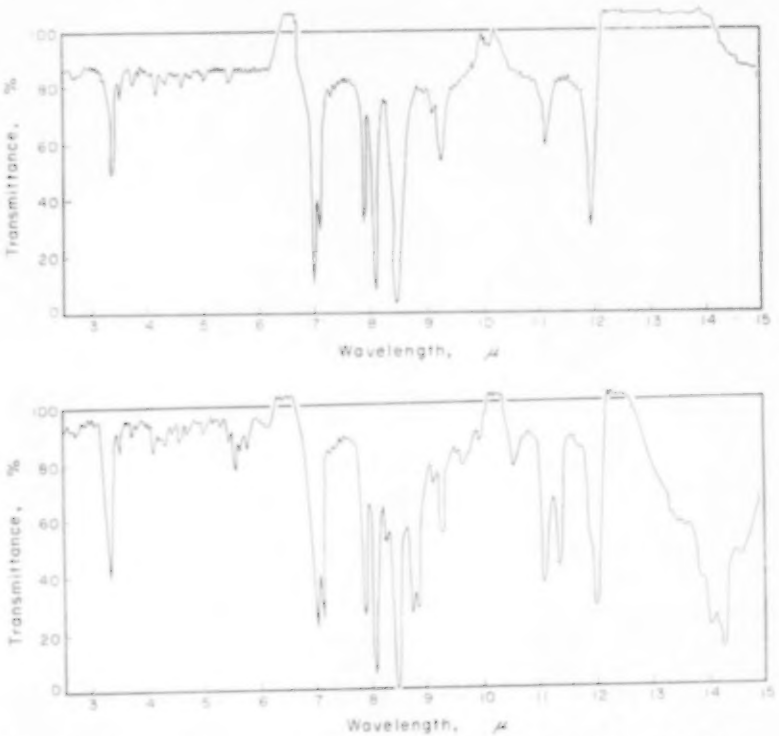


FIG. 5. Infrared spectra of 1,2-dibromoethane (upper); material desorbed from brominated carbon exposed to ethylene (lower).

After completion of the run, oven heat is discontinued, each trap is isolated by closing all of the trap stopcocks, the pumps are shut off and the system is allowed to return to atmospheric pressure. The coolants are removed and the traps allowed to warm to room temperature, or until all the contents have melted and run down to the bottom. The lower part of each trap is removed (*caution*: may be above atmospheric pressure) and to it are added: (a) a measured quantity of pure carbon tetrachloride and (b) dilute aqueous potassium hydroxide in quantity sufficient to neutralize any bromine or hydrobromic acid. The mixture is now removed, the organic layer is dried and used for spectrometric or chromatographic analysis.

ANALYSIS OF DESORPTION PRODUCT

The desorption of material from brominated carbon always gave, in addition to bromine, some aqueous hydrobromic acid. This material was produced largely by hydrolysis of bromine, rather than from dehydrobromination of organic bromides. In support of this affirmation, it was found that rigorously dried (vacuum degassed) carbon, brominated and ethylenated in the system under anhydrous conditions, and then desorbed into the cold traps, gave substantially no hydrogen bromide.

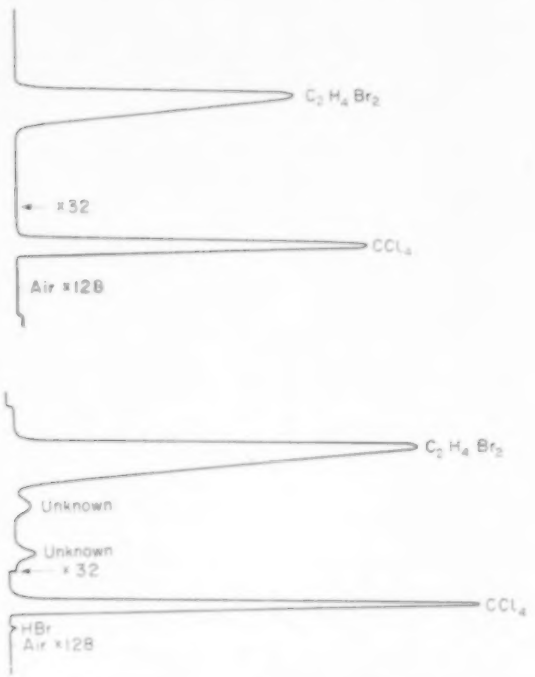


FIG. 6. Gas chromatogram of reagent dibromoethane (upper) and desorbate from brominated carbon exposed to ethylene (lower).

FIG. 5 presents infrared spectra of reagent 1,2-dibromoethane and of material desorbed from brominated carbon exposed to ethylene. The latter spectrum shows absorption in all the bands expected for the pure material, and at some additional wave lengths (8·2, 8·65, 8·75, 11·35 microns) which indicate the presence of a relatively small quantity of foreign material. The gas chromatograms in FIG. 6 confirm the identity of the desorbed material with 1,2-dibromoethane and the presence of a minor quantity of other substance (two components).

The method lends itself to quantitative estimation by establishing calibration curves from infrared or gas chromatographic peak heights or areas for various solutions of 1,2-dibromoethane in carbon tetrachloride. By such methods, the distribution of desorbed products between traps *A* and *B* for various desorption conditions was obtained; the information appears in TABLE 1. It is seen that, at oven temperature near

VOL. 5 1961

200° C, all of the rapidly recoverable material is found in the first refrigerated trap. Quantitative overall yields of 1,2-dibromoethane, based on ethylene, were obtained in many runs, although results were sometimes unaccountably erratic and some yields were as low as 45 per cent. There was no evidence that aging of the carbon sample prior to desorption, for periods up to 20 days, or that humidification of the carbon, invalidated the method.

TABLE 1. DESORPTION OF 1, 2-DIBROMOETHANE FROM BROMINATED CARBON

Sample	Molar Ratio (Br_2/C_2H_4)	Degree of Bromination of Carbon (% based on C)	Highest Desorption Temp. (° C)	Duration of Desorption (hr.)	Proportion of Yield in Trap (%)
Trap A	3·2	14·7	100	0·5	3
Trap B	3·2	14·7	360	3·5	97
Trap A	3·1	14·5	140	0·7	40
Trap B			203	5·3	60
Trap A	2·7	12·4	213	0·5	100
Trap B			352	4·8	0
Trap A	4·5	9·0	212	2·0	100
Trap B			245	2·3	0

IDENTIFICATION OF HYDROCARBONS IN OUTDOOR AIR

An outdoor sampling unit was constructed of commercially available components, as shown in FIG. 7. An integral motor-blower-canister unit (Barnebey-Cheney Company, Columbus, Ohio) was selected; the canister was made up of two concentric cylinders of perforated stainless steel, the annular space between them being filled with activated coconut shell carbon. Canister dimensions were: 6 in. height; 3·5 in. o.d.; 0·75 in. bed thickness. To the output of the motor-driven blower, normally unobstructed, another canister was attached, having identical dimensions as the first, but filled with 10 per cent brominated carbon. The path of the sampled air was therefore through activated carbon, then through brominated activated carbon, and then returned to atmosphere. Total air flow was 10 ft^3 per min. This sampler was set into operation at a monitor station of the Department of Air Pollution Control of the City of New York, at East 59th Street, with the co-operation of the Department. Vehicular traffic and a bus garage are in the vicinity of the station.

After operation for one month, vacuum stripping of the first, or activated carbon, stage gave a sample whose infrared spectrum indicated generally that organic matter had been collected; no attempt at separation or identification of components was made. Desorption from the brominated carbon stage, and gas chromatographic examination of the desorbate, showed the peak corresponding to dibromoethane, with absence of side peaks. The qualitative identification of atmospheric ethylene is thereby confirmed.

FIG 7. Outdoor sampling unit.

VOL.
5
1961

CONCLUSION

It is inescapable to infer that the identification of 1,2-dibromoethane, removed from a sample of brominated carbon, which has been exposed to an air stream previously passed through activated carbon, demonstrates the presence of ethylene in the air stream.

It is recognized that yield data based on relatively large ethylene concentrations cannot be used to predict performance in natural atmospheres containing ethylene in the "parts per billion" range. Such extreme dilutions cannot be simulated reliably in test chambers because wall adsorption errors become high. For validation of ethylene analyses in outdoor atmospheres, a study of the relation between the amount of ethylene obtained by the brominated carbon method and the degree of dried sepal injury to orchids (an ethylene effect) has been initiated and will be described in a subsequent report.

In any event, the scheme described here presents a new and potentially useful approach to atmospheric sampling for ethylene, which permits simultaneous routine adsorptive collection of those heavier organic vapors which accumulate on and can be recovered from unimpregnated activated carbon.

Reactive adsorptive sampling of this type can be developed by the exploitation of other chemical transformations (e.g., chlorination, hydrobromination, ozonization) which may be analytically useful.

Acknowledgement—This work was entirely supported by a Research Grant of the National Institutes of Health, Public Health Service.

VOL.
5
1961

REFERENCES

Gross C. R. unpublished work. Cornell University.

Ray N. H. (1955) The Separation and Estimation of Volatile Organic Compounds by Gas-Liquid Partition Chromatography. *Analyst* **80**, 853.

Smith S. B. (1959) 136th Meeting, *Amer. Chem. Soc.* Atlantic City, N.J.

Turk A. (1960) *Odor Measurement and Control*, Air Pollution Abatement Manual, Chap. 13. Mfg. Chem. Assn., Washington, D.C.

Turk A., Sleik H. and Messer P. J. (1952) Determination of Gaseous Air Pollution by Carbon Adsorption. *Amer. Ind. Hygiene Assn. Quart.* **13**, 23.

West P. W., Sen B. and Gibson N. A. (1958) Gas-Liquid Chromatography Analysis Applied to Air-Pollution. *Anal. Chem.* **30**, 1390.

Wirth H. (1953) Separation of gases. *Monatsh.* **84**, 751.

Int. J. Air Wat. Poll. Pergamon Press 1961. Vol. 5, No. 1, pp. 24–33. Printed in Great Britain.

THE APPLICATION OF A COPPER-SILICA CATALYST FOR THE REMOVAL OF NITROGEN OXIDES PRESENT IN LOW CONCENTRATIONS BY CHEMICAL REDUCTION WITH CARBON MONOXIDE OR HYDROGEN*

S. SOURIRAJAN† and J. L. BLUMENTHAL

(*Received* 9 *January* 1961)

Abstract—The catalytic reduction reactions of NO and NO_2 with carbon monoxide and hydrogen have been studied from the point of view of their importance to air pollution control. It was found that with a 5–10 fold excess of carbon monoxide, hydrogen or both, more than 90 per cent of NO and NO_2 was removed by chemical reactions at 300° C at a gas space velocity of 10,000 hr^{-1}, in the presence of the copper-silica catalyst. The inclusion of hydrocarbons in the reactants seemed to aid the reduction reactions and promote the effectiveness of the catalyst for the removal of nitrogen oxides. The presence of traces of oxygen in the reactants containing an excess of the reducing gases did not affect the efficiency of the catalyst. The possible practical application of the copper oxide catalyst for the removal of nitrogen oxides hydrocarbons and carbon monoxide present in the auto exhaust gases, has been pointed out; for this purpose, a two stage catalytic converter system has been suggested, the catalyst in the first stage being copper in the reducing atmosphere of the exhaust gases, and the catalyst in the second stage being copper oxide in the oxidizing atmosphere of the exhaust gases and secondary air.

INTRODUCTION

THE following reduction reactions of NO and NO_2 with carbon monoxide or hydrogen are particularly important from the point of view of air pollution control, since they suggest a possible means of removal of nitrogen oxides present in low concentrations in the auto exhaust gases.

Reaction	ΔF°_{298} (in calories)	ΔH°_{298} (in calories)
$NO + CO \longrightarrow CO_2 + \frac{1}{2}N_2$	−82, 180	−89, 250
$NO + H_2 \longrightarrow H_2O + \frac{1}{2}N_2$	−75, 359	−79, 400
$NO + \frac{5}{2}H_2 \longrightarrow NH_3 + H_2O$	−79, 238	−90, 360
$NO_2 + 2CO \longrightarrow 2CO_2 + \frac{1}{2}N_2$	−134, 670	−143, 160
$NO_2 + 2H_2 \longrightarrow 2H_2O + \frac{1}{2}N_2$	−121, 000	−123, 000
$NO_2 + \frac{7}{2}H_2 \longrightarrow NH_3 + 2H_2O$	−126, 200	−134, 520

The equilibrium constants (K_a) for the above reactions, as well as for the decomposition reactions of NO and NO_2, in the temperature range 298°–1800° K, are given in

*Contribution from the Department of Engineering, University of California, Los Angeles 24, California.

†Present address: Institute of Geophysics, University of California, Los Angeles 24, California.

FIG. 1. These thermodynamic data show clearly that all the above reactions are feasible and are favored by lower temperatures. Such conditions are ideally suited for the use of catalysts to accomplish the effective removal of nitrogen oxides present in low concentrations, by the above reactions.

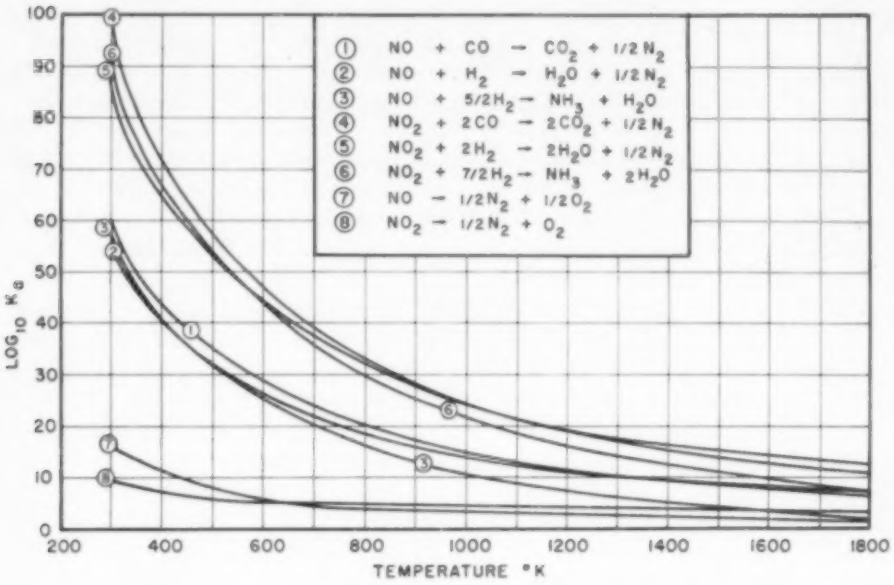


FIG. 1. Equilibrium constants (K_a) for the reduction and decomposition reactions of nitrogen oxides.

TAYLOR (1959) demonstrated the feasibility of the removal of nitric oxide present in low concentrations by reduction reactions with carbon monoxide and/or hydrogen in the presence of catalysts such as zinc-copper chromite, iron chromite, barium promoted copper chromite and chromium promoted iron oxide. Experiments with copper oxide-silica ($CuO\text{-}SiO_2 = 30 : 70$) catalyst in conjunction with 700–1000 parts per million (p.p.m.) of nitric oxide mixed with the exhaust gases from a two-cylinder gasoline engine were found to result in almost complete removal of NO at about 350° C (SOURIRAJAN and BLUMENTHAL, 1960c). These latter observations were particularly interesting in view of the proven effectiveness of the copper oxide catalyst for the total oxidation of hydrocarbons and carbon monoxide present in auto exhaust gases (SOURIRAJAN *et al.*, 1960a, 1960b, 1961). Consequently the object of this investigation was to study the applicability of the copper-silica catalyst for the removal of NO and NO_2 present in low concentrations, by the chemical reduction reactions with carbon monoxide and/or hydrogen in the presence or absence of hydrocarbons.

EXPERIMENTAL DETAILS

The apparatus used was similar to the one described earlier (SOURIRAJAN and BLUMENTHAL, 1960c). It consisted essentially of a gas sample tank containing the required concentration of reactant gases in diluent nitrogen under pressure, a Vycor

VOL. 5 1961

glass preheater and reactor tube, 80 cm long and 2 cm diameter, enclosed in a furnace, and a gas sampling and gas analysis system. NO and NO_2 concentrations in the range 400–1400 p.p.m. along with 1 to 10 times the amount of the reducing agent, carbon monoxide and/or hydrogen, were used in these experiments. The effect of the presence of hydrocarbons and traces of oxygen on the performance of the catalyst was tested by admixing n-hexane and air in the reactants.

The concentrations of NO and NO_2 in the gases entering and leaving the catalyst bed were determined by oxidizing the NO contained in a sample of the gas to NO_2 by excess air, and estimating the total NO_2 in the sample by using the Saltzman reagent (SALTZMAN, 1954).

The gas sampling and gas analysis system included in the apparatus consisted of the following: (i) a fine capillary through which the gas sample was drawn by vacuum suction, (ii) a rotameter to measure the sample gas flow rate into the system, (iii) another capillary through which atmospheric air was drawn into a glass chamber containing an ozone lamp and (iv) a succession of five bubblers containing 20 cm^3 of Saltzman reagent to estimate the NO_2 present in the sample gas drawn. The ozone lamp generated extremely small quantities of ozone which facilitated the complete oxidation of NO to NO_2. By maintaining a constant differential pressure between the atmosphere and ozone chamber, constant volume flow rates through the sample-gas and air capillaries were obtained. The capillaries were initially calibrated, and in this work, a sample-gas flow rate of 9–16 cm^3/min and an air flow rate of 180–200 cm^3/min were used. In each experiment the gas sample for analysis was drawn for five to ten minutes. The above method of analysis was previously compared with the phenoldisulfonic acid method for the estimation of total oxides of nitrogen (ELKINS, 1959), and the results were found to check well within ± 1 per cent. The gas sampling and the gas analysis methods used in this work were found to give reproducible results within 1 per cent.

25 cm^3 of the copper oxide-silica ($CuO : SiO_2 = 30 : 70$) catalyst described earlier (SOURIRAJAN and BLUMENTHAL, 1960c) was used in these investigations. The catalyst was prepared as follows. The required quantity of a dilute aqueous solution of copper nitrate was impregnated with the appropriate amount of the wet silica gel and then evaporated slowly on a water bath to a suitable consistency which was finally extruded. The extruded catalyst mass was dried to 110° C and heated in a current of air at 400° C for 12 hours. This catalyst had a surface area of 227 m^2/g, a pore volume of 0·70 cm^3/g and an average pore diameter of 62Å. Prior to use in the actual experiments, the above copper oxide-silica catalyst was reduced in a current of hydrogen for 12 hours at 400° C. Thus the active catalyst for the reduction reactions of NO and NO_2 was reduced copper.

EXPERIMENTAL RESULTS AND DISCUSSION

The experimental data on the extent of removal of NO (400–800 p.p.m. in the reactants) or NO_2 (550–1400 p.p.m. in the reactants) by reduction reactions with carbon monoxide and/or hydrogen from stoichiometric quantities up to ten fold excess in the presence or absence of a hydrocarbon (namely, n-hexane) and oxygen, are given in TABLE 1. These data cover the temperature range 83°–536° C at gas space velocities (volume of gas at 25° C and atmospheric pressure per hour per volume of

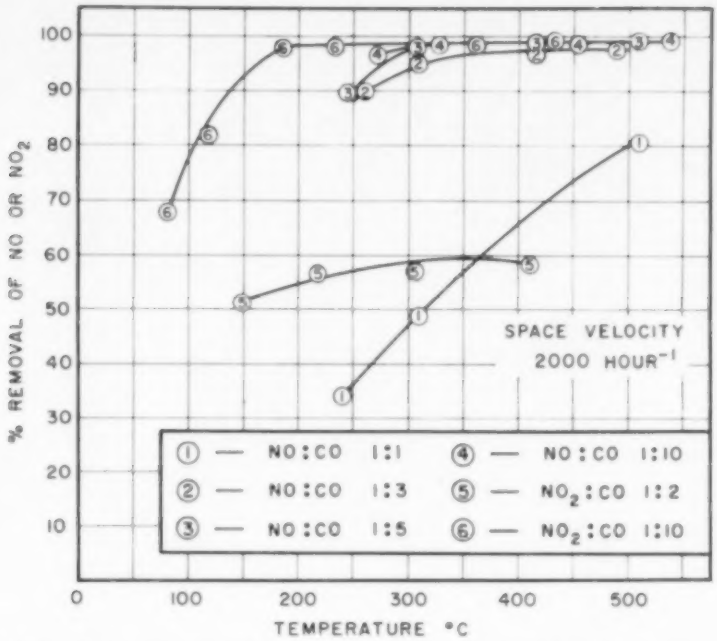


FIG. 2. Removal of nitrogen oxides by reaction with carbon monoxide—effect of temperature.

VOL.
5
1961

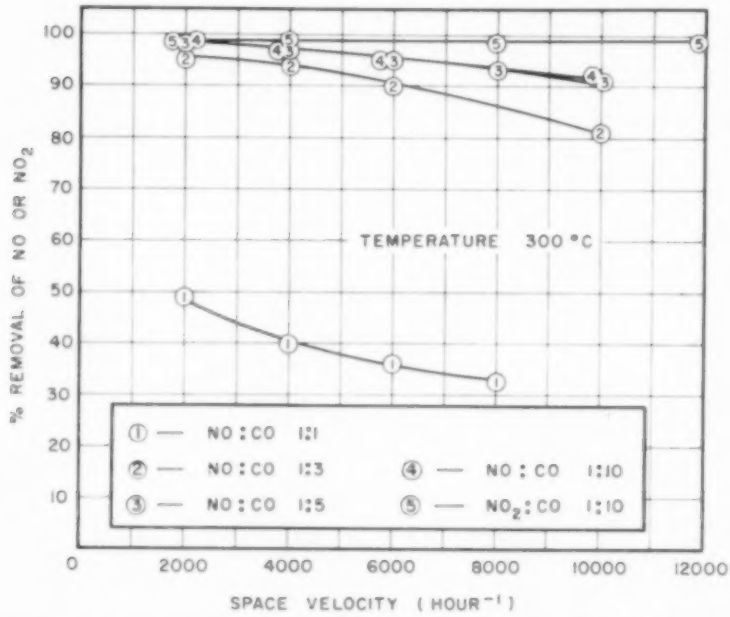


FIG. 3. Removal of nitrogen oxides by reaction with carbon monoxide—effect of space velocity.

TABLE I. REMOVAL OF NITROGEN OXIDES PRESENT IN LOW CONCENTRATIONS BY CHEMICAL REDUCTION WITH CARBON MONOXIDE OR HYDROGEN IN THE PRESENCE OF THE COPPER-SILICA CATALYST

Run No.	Temperature °C	Space velocity hour^{-1}	NO p.p.m.	NO_2 p.p.m.	CO p.p.m.	H_2 p.p.m.	n-hexane p.p.m.	oxygen p.p.m.	Percent Removal of NO or NO_2
1	240	2,000	700	–	750	–	–	–	34
2	300	2,000	700	–	750	–	–	–	49
3	446	2,000	700	–	750	–	–	–	53
4	510	2,000	700	–	750	–	–	–	81
5	300	4,000	700	–	750	–	–	–	40
6	300	6,000	670	–	750	–	–	–	37
7	300	8,000	620	–	750	–	–	–	32
8	249	2,000	410	–	1,200	–	–	–	88
9	300	2,000	410	–	1,200	–	–	–	95
10	414	2,000	410	–	1,200	–	–	–	97
11	500	2,000	410	–	1,200	–	–	–	97
12	300	4,000	370	–	1,200	–	–	–	95
13	300	6,000	360	–	1,200	–	–	–	90
14	300	10,000	360	–	1,200	–	–	–	81
15	244	2,000	510	–	2,500	–	–	–	88
16	300	2,000	510	–	2,500	–	–	–	98
17	415	2,000	510	–	2,500	–	–	–	98
18	510	2,000	545	–	2,500	–	–	–	98
19	300	4,000	545	–	2,500	–	–	–	96
20	300	6,000	545	–	2,500	–	–	–	95
21	300	8,000	545	–	2,500	–	–	–	93
22	300	10,000	545	–	2,500	–	–	–	91
23	270	2,000	680	–	6,800	–	–	–	97
24	300	2,000	680	–	6,800	–	–	–	98
25	453	2,000	680	–	6,800	–	–	–	98
26	536	2,000	680	–	6,800	–	–	–	98
27	300	4,000	670	–	6,800	–	–	–	96
28	300	6,000	670	–	6,800	–	–	–	95
29	300	10,000	670	–	6,800	–	–	–	92
30	148	2,000	–	559	1,100	–	–	–	51
31	215	2,000	–	559	1,100	–	–	–	56
32	304	2,000	–	559	1,100	–	–	–	57
33	410	2,000	–	559	1,100	–	–	–	58
34	434	2,000	–	1,400	14,000	–	–	–	98
35	360	2,000	–	1,400	14,000	–	–	–	98
36	300	2,000	–	1,400	14,000	–	–	–	98
37	232	2,000	–	1,400	14,000	–	–	–	98
38	184	2,000	–	1,400	14,000	–	–	–	98
39	115	2,000	–	1,400	14,000	–	–	–	82
40	83	2,000	–	1,400	14,000	–	–	–	67
41	300	4,000	–	1,400	14,000	–	–	–	98
42	300	8,000	–	1,400	14,000	–	–	–	98
43	300	12,000	–	1,400	14,000	–	–	–	98
44	235	2,000	650	–	6,500	–	3,000	–	98
45	300	2,000	650	–	6,500	–	3,000	–	98
46	374	2,000	650	–	6,500	–	3,000	–	98
47	452	2,000	650	–	6,500	–	3,000	–	98
48	300	4,000	650	–	6,500	–	3,000	–	98
49	300	6,000	650	–	6,500	–	3,000	–	98
50	300	10,000	650	–	6,500	–	3,000	–	98

Run No.	Temperature °C	Space velocity hour^{-1}	NO p.p.m.	NO_2 p.p.m.	CO p.p.m.	H_2 p.p.m.	n-hexane p.p.m.	oxygen p.p.m.	Percent Removal of NO or NO_2
51	240	2,000	790	–	7,900	7,900	3,000	–	98
52	300	2,000	790	–	7,900	7,900	3,000	–	98
53	414	2,000	790	–	7,900	7,900	3,000	–	98
54	485	2,000	790	–	7,900	7,900	3,000	–	98
55	300	4,000	580	–	5,800	5,800	3,000	–	98
56	300	8,000	580	–	5,800	5,800	3,000	–	98
57	300	12,000	580	–	5,800	5,800	3,000	–	98
58	112	2,000	–	610	6,100	6,100	3,000	–	90
59	262	2,000	–	610	6,100	6,100	3,000	–	98
60	300	2,000	–	610	6,100	6,100	3,000	–	98
61	428	2,000	–	610	6,100	6,100	3,000	–	98
62	300	4,000	–	610	6,100	6,100	3,000	–	98
63	300	8,000	–	610	6,100	6,100	3,000	–	98
64	300	12,000	–	610	6,100	6,100	3,000	–	98
65	300	6,000	–	785	7,850	7,850	3,000	785	98
66	300	10,000	–	785	7,850	7,850	3,000	785	98
67	300	10,000	–	785	7,850	7,850	3,000	785	98
68	280	2,000	530	–	–	530	–	–	91
69	300	2,000	530	–	–	530	–	–	94
70	414	2,000	530	–	–	530	–	–	95
71	486	2,000	530	–	–	530	–	–	95
72	300	4,000	460	–	–	530	–	–	85
73	300	6,000	460	–	–	530	–	–	78
74	300	10,000	460	–	–	530	–	–	72
75	252	2,000	480	–	–	1,260	–	–	92
76	300	2,000	420	–	–	1,260	–	–	95
77	484	2,000	420	–	–	1,260	–	–	98
78	510	2,000	420	–	–	1,260	–	–	98
79	300	4,000	400	–	–	1,260	–	–	90
80	300	6,000	400	–	–	1,260	–	–	87
81	300	8,000	400	–	–	1,260	–	–	80
82	259	2,000	530	–	–	2,500	–	–	98
83	300	2,000	530	–	–	2,500	–	–	98
84	446	2,000	590	–	–	2,500	–	–	98
85	500	2,000	590	–	–	2,500	–	–	98
86	300	4,000	500	–	–	2,500	–	–	96
87	300	6,000	520	–	–	2,500	–	–	91
88	300	8,000	520	–	–	2,500	–	–	89
89	94	2,000	750	–	–	7,500	–	–	45
90	156	2,000	750	–	–	7,500	–	–	98
91	205	2,000	750	–	–	7,500	–	–	98
92	300	2,000	750	–	–	7,500	–	–	98
93	300	8,000	750	–	–	7,500	–	–	98
94	300	12,000	750	–	–	7,500	–	–	98
95	260	2,000	800	–	–	8,000	3,000	–	94
96	300	2,000	800	–	–	8,000	3,000	–	98
97	415	2,000	800	–	–	8,000	3,000	–	98
98	486	2,000	800	–	–	8,000	3,000	–	98
99	300	4,000	800	–	–	8,000	3,000	–	98
100	300	6,000	800	–	–	8,000	3,000	–	98
101	300	10,000	800	–	–	8,000	3,000	–	95

VOL. 5 1961

Run No.	Temperature °C	Space velocity hour^{-1}	NO p.p.m.	NO_2 p.p.m.	CO p.p.m.	H_2 p.p.m.	n-hexane p.p.m.	oxygen p.p.m.	Percent Removal of NO or NO_2
102	100	2,000	–	1,390	–	13,900	–	–	45
103	147	2,000	–	1,390	–	13,900	–	–	86
104	204	2,000	–	1,390	–	13,900	–	–	98
105	300	2,000	–	1,390	–	13,900	–	–	98
106	300	4,000	–	1,390	–	13,900	–	–	98
107	300	6,000	–	1,390	–	13,900	–	–	98
108	300	10,000	–	1,390	–	13,900	–	–	98
109	168	2,000	–	1,320	–	1,320	–	–	61
110	242	2,000	–	1,320	–	1,320	–	–	76
111	326	2,000	–	1,320	–	1,320	–	–	79
112	400	2,000	–	1,320	–	1,320	–	–	98

catalyst bed) in the range 2000–12,000 hr^{-1}. The effects of temperature and gas space velocity on the extent of removal of NO or NO_2 by reactions with carbon monoxide are summarized in FIGS. 2 and 3, and those by the reactions involving hydrogen are summarized in FIGS. 4 and 5.

It was found that with a 5–10 fold excess of carbon monoxide, hydrogen or both, either in the presence or absence of the hydrocarbon, more than 90 per cent of NO and NO_2 was removed by the catalytic reduction reactions at 300° C at a gas space velocity of 10,000 hr^{-1}. With lower proportions of the reducing agent, the extent of removal of the nitrogen oxides was somewhat less apparently due to the partial oxidation of the copper catalyst. The presence of the hydrocarbons in the reactants seemed to aid the reduction reactions and promote the effectiveness of the catalyst for the removal of nitrogen oxides. The presence of traces of oxygen in the reactants containing an excess of the reducing gases, did not affect the efficiency of the catalyst at all.

When hydrogen was reacting with the nitrogen oxides, the exhaust gases from the reactor were found to contain considerable amounts of ammonia. The extent of conversion of NO and NO_2 into ammonia by reactions with hydrogen in the presence of the copper catalyst was quantitatively estimated by making use of the Nessler's reagent (SNELL and SNELL, 1941a) for a few typical runs. The results obtained are given in TABLE 2. The results showed that, depending on the experimental conditions, 20 to 98 per cent of nitrogen oxides could be converted into ammonia by reactions with hydrogen. Should this be an undesirable feature from the point of view of air pollution control from any particular source, it would seem possible to modify the nature of the copper catalyst so as to retard the ammonia forming reactions.

When carbon monoxide, hydrocarbons and/or hydrogen were reacting with the nitrogen oxides, the exhaust gases from the reactor were also analysed for the presence of HCN (SNELL and SNELL, 1941b). No trace of HCN was found to form in the reactions.

The above results show that reduced copper is an effective catalyst for the reduction reactions of NO and NO_2 with carbon monoxide or hydrogen, and that traces of oxygen do not poison the catalyst in the above reactions. Hence it is not necessary to remove all traces of oxygen from the reactants before the nitrogen oxides could be

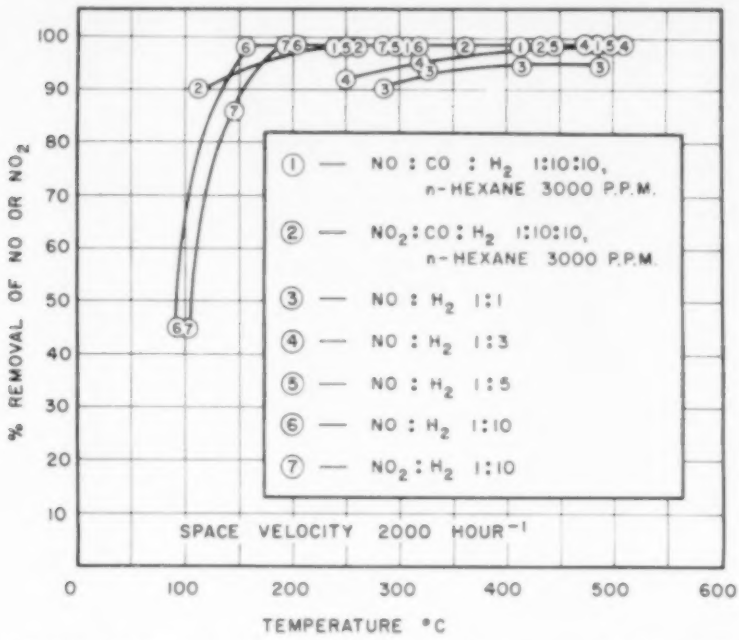

% REMOVAL OF NO OR NO2
TEMPERATURE °C
SPACE VELOCITY 2000 HOUR⁻¹
① — NO : CO : H2 1:10:10, n-HEXANE 3000 P.P.M.
② — NO2 : CO : H2 1:10:10, n-HEXANE 3000 P.P.M.
③ — NO : H2 1:1
④ — NO : H2 1:3
⑤ — NO : H2 1:5
⑥ — NO : H2 1:10
⑦ — NO2 : H2 1:10

FIG. 4. Removal of nitrogen oxides by reactions involving hydrogen—effect of temperature.

VOL. 5 1961

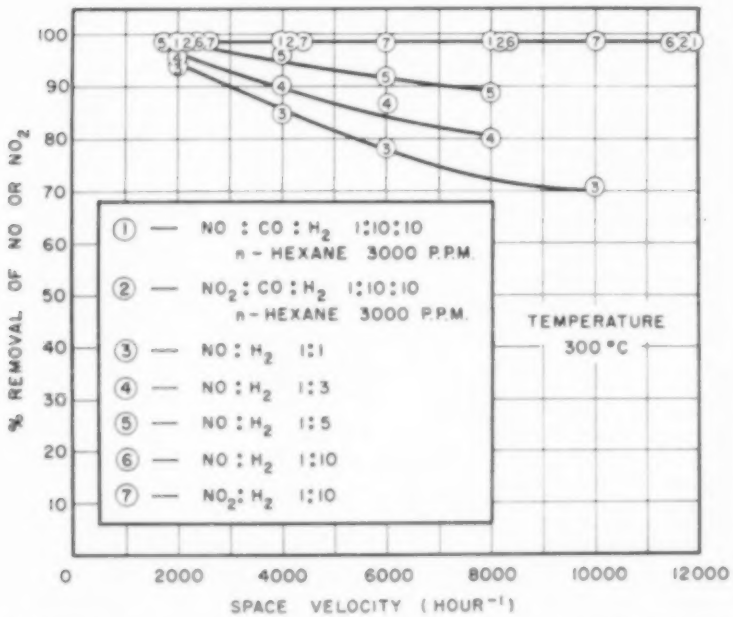

% REMOVAL OF NO OR NO2
SPACE VELOCITY (HOUR⁻¹)
TEMPERATURE 300 °C
① — NO : CO : H2 1:10:10 n-HEXANE 3000 P.P.M.
② — NO2 : CO : H2 1:10:10 n-HEXANE 3000 P.P.M.
③ — NO : H2 1:1
④ — NO : H2 1:3
⑤ — NO : H2 1:5
⑥ — NO : H2 1:10
⑦ — NO2 : H2 1:10

FIG. 5. Removal of nitrogen oxides by reactions involving hydrogen—effect of space velocity.

TABLE 2. FORMATION OF AMMONIA BY THE REDUCTION OF NITROGEN OXIDES WITH HYDROGEN IN THE PRESENCE OF THE COPPER-SILICA CATALYST

Run No.	Temperature (°C)	Space velocity (hr^{-1})	NO (p.p.m.)	NO_2 (p.p.m.)	CO (p.p.m.)	H_2 (p.p.m.)	n-hexane (p.p.m.)	Per cent removal of NO or NO_2	Per cent conversion of NO or NO_2 to ammonia
113	300	2,000	900	–	9,000	9,000	3,000	98	85
114	148	2,000	900	–	9,000	9,000	3,000	98	58
115	300	2,000	640	–	–	6,400	–	98	98
116	228	2,000	640	–	–	6,400	–	98	98
117	300	2,000	–	487	–	4,870	–	98	98
118	150	2,000	–	487	–	4,870	–	98	65
119	460	2,000	660	–	6,600	1,320	–	98	88
120	240	2,000	660	–	6,600	1,320	–	98	74
121	190	2,000	660	–	6,600	1,320	–	98	65
122	154	2,000	580	–	5,800	580	–	98	0
123	204	2,000	580	–	5,800	580	–	98	32
124	500	2,000	580	–	5,800	580	–	98	34
125	150	2,000	–	520	5,200	520	–	98	22
126	300	2,000	–	520	5,200	520	–	98	41
127	450	2,000	–	520	5,200	520	–	98	45
128	300	10,000	–	520	5,200	520	–	98	41

successfully reduced. For the efficient removal of NO or NO_2 by the catalytic reduction reactions with carbon monoxide or hydrogen, it is only necessary that the copper catalyst be in a reducing atmosphere all the time, and a sufficient excess of the reducing agent be present in the reactor. This is not a problem particularly in the auto exhaust gases in which the NO concentration range is 10–3000 p.p.m. and the CO concentration range is 15,000–70,000 p.p.m. Thus under most conditions of practical operation of an auto engine, more than sufficient excess of carbon monoxide is present for almost complete removal of NO by the catalytic reduction reaction.

CONCLUSION

Thus the copper oxide catalyst developed in this program of work (SOURIRAJAN *et al.*, 1960a, 1960b, 1960c, 1961) promises to be a very practical catalyst for the removal of nitrogen oxides, hydrocarbons and carbon monoxide present in the auto exhaust gases. For this application, one would visualize a two stage converter system, the catalyst in the first stage being copper (copper oxide initially reduced) in the reducing atmosphere of the exhaust gases, and the catalyst in the second stage being copper oxide in the oxidizing atmosphere of the exhaust gases and the secondary air. A temperature of only 300° C would be needed in the first stage wherein more than 90 per cent of NO could be removed by the catalytic reduction reaction with the carbon monoxide present in the exhaust gases. A good proportion of the remaining NO, and almost all of the hydrocarbons and carbon monoxide could be removed in the second stage wherein the catalyst temperature should be kept at or above 400° C as discussed earlier.

Acknowledgements—This investigation forms part of the catalysis studies in the air pollution research program being supported by the State of California. The authors are grateful to Professors L. M. K. BOELTER, W. C. HURTY and J. M. ENGLISH for their continued support and encouragement to this work. Our thanks are due to Mr. FRED MILSTEIN for his valuable assistance in this project and to Mr. DELMAR J. ALBRIGHT who fabricated the glass apparatus.

REFERENCES

ELKINS H. B. (1959) *The Chemistry of Industrial Toxicology* p. 378. John Wiley, New York.

SALTZMAN B. E. (1954) Colorimetric microdetermination of nitrogen dioxide in the atmosphere. *Anal. Chem.* **26,** 1949–54.

SNELL F. D. and SNELL C. T. (1941a) *Colorimetric Methods of Analysis, Volume* I, pp. 649–659. Van Nostrand, New York.

SNELL F. D. and SNELL C. T. (1941b) *Colorimetric Methods of Analysis, Volume* I, pp. 125–129. Van Nostrand, New York.

SOURIRAJAN S. and ACCOMAZZO M. A. (1960a) Catalytic oxidation of Carbon monoxide present in low concentrations. *Canad. J. Chem. Engng.* **38,** 1990–98.

SOURIRAJAN S. and ACCOMAZZO M. A. (1961) The application of the copper oxide-alumina catalyst for air pollution control. *Canad. J. Chem. Engng.* **39** 88-93

SOURIRAJAN S., ACCOMAZZO M. A. and KEN NOBE (1960b) Catalytic oxidation of n-hexane present in low concentrations. *Proceedings of the Second International Congress on Catalysis* Paper **130.** Editions Technip, Paris.

SOURIRAJAN S. and BLUMENTHAL J. L. (1960c) Catalytic decomposition of nitric oxide present in low concentrations. *Proceedings of the Second International Congress on Catalysis* Paper **131.** Editions Technip, Paris.

TAYLOR F. R. (1959) Elimination of oxides of nitrogen from automobile exhaust. *Report No. 28,* 1–49. Air Pollution Foundation, San Marino, California.

VOL.
5
1961

Int. J. Air Wat. Poll. Pergamon Press 1961. Vol. 5, No. 1, pp. 34–45. Printed in Great Britain.

THE EFFECT OF SOLID ATMOSPHERIC POLLUTANTS UPON THE TRANSMISSION OF A COLLIMATED BEAM OF INFRA-RED RADIATION

W. L. DENNIS

Chemical Defence Experimental Establishment, Porton, Wilts.

(*Received* 23 *January* 1960)

Abstract—Published work on the absorption and scattering of infra-red radiation in a collimated beam by the gases of the atmosphere, water vapour and haze is reviewed and the correlation between visual range, concentration of suspended matter and transmission is discussed.

In view of the inadequacy of data on the effect of atmospheric pollution (dust and smoke), a technique has been developed for collecting a representative sample on a millipore filter, for determination of concentration by weighing, and for transferring the deposit to a sodium chloride disc for transmission measurements with an infra-red spectrometer. A series of curves relating transmission at various wavelengths, over the range from 1 to 10 microns, to concentration of pollution has been derived. The relationship between visual range and concentration has also been determined approximately.

To illustrate the application of the results, curves have been produced showing the transmission, corresponding to various visual ranges, as effected by atmospheric pollution alone for wavelengths from 1 to 10 microns over a path length of 10·1 miles in comparison with the curve for water vapour and CO_2 in a clear atmosphere at 24 miles visual range.

INTRODUCTION

A CONSIDERABLE amount of research has been carried out in recent years on the transmission of infra-red radiation through the atmosphere. The problem is very complex because of its variable composition. This applies especially to the amount of water vapour, which is the main attenuating constituent. Besides attenuation by molecular absorption and scattering, radiation is lost by scattering by haze particles and by fog and cloud droplets.

The water vapour and carbon dioxide present in the atmosphere give rise to selective molecular absorption in the infra-red region but there are windows, or regions of relatively high transmission between the strong absorption bands. In addition throughout the spectrum there is a continuous attenuation due to fog, haze etc. ELDER and STRONG (1953) derived an empirical equation, based upon experimental data from a number of different sources, which relates the transmission coefficients of these windows to the amount of precipitable water in the path, i.e. the length of the column if all the vapour were condensed to a liquid. The other main absorber, carbon dioxide, remains sensibly constant at about 0·3 per cent by volume. Absorption, due to the rarer constituents of the air and molecular scattering above a wavelength of about 1μ, is generally negligible in comparison with the other processes.

Tables which enable the transmittance through atmospheric water vapour and carbon dioxide to be calculated for various path lengths have been computed by LARMORE (1956) assuming an error function law of attenuation.

SCATTERING BY HAZE

Some of the most reliable experimental transmission data are those given by GEBBIE, HARDING, HILSUM, PRYCE and ROBERTS (1951) who measured the absolute transmissions in the 1 to 14μ region over two horizontal paths by comparing the transmissions at four different wavelengths at the peaks of the window regions with that at 0·61μ, the transmission at this wavelength being measured photometrically. Their results agree with an exponential law of attentuation with distance. They showed that assuming the scattering was by salt nuclei, which changed in size and refractive index with humidity, the size distribution and concentration deduced from their data were in agreement with values quoted in the literature.

LOCKE and collaborators (1956) have used these results and derived curves showing the variation in the transmission of a 2000 yd path with wavelength (0·05 to 3·6μ) for given states of haze as specified by the visual ranges and with a water vapour content of 17 mm of precipitable water. These curves were obtained by plotting the transmissions in several window regions so that although the effect of the water vapour is not eliminated it is reduced to a minimum. TABLE 1 is calculated from data given in their paper.

TABLE 1.

Visual Range (Statute miles)	Percentage Transmission per statute mile			
	$\lambda = 1\mu$	$\lambda = 2\mu$	$\lambda = 3\mu$	$\lambda = 4\mu$
5	55	68	77	80
10	72	81	88	94
15	80	86	93	96

VOL. 5 1961

YATES (1951) has computed the atmospheric transmission of near infra-red radiation as a function of the water vapour and carbon dioxide content, the visual transmission at 0·61μ, the path length and the black body source temperature.

He also derived the transmission of the haze from the results of GEBBIE *et al.* (1951). Their data showed that beyond 7μ there is a sharp decrease in the transmission due to weak bands of water vapour and the wings of rotation bands lying further out. To separate the effects of the haze from those due to the water vapour at these longer wavelengths YATES plotted the infra-red transmission against the visual transmission and found that as the visibility increased, the transmissions approached values in good agreement with those computed by HULBERT (1944) for the effects of water vapour alone. He therefore assumed that at these wavelengths any departure from Hulbert's values is due to the haze present.

A similar conclusion was reached by ROACH and GOODY (1958) who undertook a series of simultaneous spectral observations of absorption and emission in the atmospheric window between 8μ and 13μ in London and the relatively clean country air at Ascot. They considered that although their results showed a correlation between the continuous extinction and water vapour in both places, aerosol was largely responsible for the extinction in London, while in the country it was caused mainly by the wings of distant lines.

DEIRMENDJIAN (1960) computed the continuous spectrum of haze extinction using water droplet size distribution models and found good agrrement with existing experimental results for the window regions and supported the conclusions of ROACH and GOODY (1958) on the importance of wing absorption in the 8μ to 12μ region. He also showed that the haze extinction is very sensitive to size distribution but increasing the overall concentration of particles results in an increase in the extinction coefficient at all wavelengths by the same factor.

VISUAL RANGE AS A MEASURE OF ATMOSPHERIC AEROSOL CONCENTRATION

As indicated above, the visual range or transmission through the atmosphere has been used by various workers to estimate the amount of suspended matter in the air. As the light scattered by a cloud of particles depends not only upon the concentration and path length but also upon the size distribution, composition and shape of the particles, this can only be done if these latter parameters are constant. Radiation scattered in the infra-red region similarly depends upon these same parameters. If, however, the size distribution and composition of the haze particles are reasonably constant, the transmission at any particular wavelength in the infra-red region can be correlated with the visual range or visual transmission.

The fraction I/I_0 of the radiation of wavelength λ transmitted through an absorbing medium of path length l and concentration c, is given by $I/I_0 = e^{-a\lambda cl}$, where $a\lambda$ is the attenuation coefficient and depends upon the wavelength. When the concentration is incorporated in the attenuation coefficient, this can be simplified to $I/I_0 = e^{-\sigma\lambda l}$ and $\sigma\lambda$ is proportional to the concentration.

From the theory of KOSCHMIEDER (1924) it can be deduced that the visual range $v = 3{\cdot}91/\sigma\lambda$ where $\sigma\lambda$ is then the attenuation coefficient corresponding to the visible region of the spectrum. Values of $\sigma\lambda$ against v are tabulated in the Smithsonian Meteorological Tables and also by STEFFENS (1956). It follows that the visual range is inversely proportional to the concentration of suspended matter in the air. STEWART (1956) has found empirically that $\sigma = 6{\cdot}5w$ Km^{-1} where w is the weight of particulate in mg/m^3 and σ the attenuation coefficient due to "smoke" in the air, from which it follows that $v = 0{\cdot}374/w$ miles. This was based on data obtained in dry weather using an Owens' automatic smoke filter. It must be borne in mind, however, that the visual range is very sensitive to particle size distribution as demonstrated by the computations of STEFFENS (1956) and DEIRMENDJIAN (1960).

EXPERIMENTAL PROCEDURE

The procedure adopted in the present investigation was to obtain daily average samples over a period of several hours in the form of deposits of discrete particles and to determine the infra-red transmission and concentration corresponding to each deposit. It was then assumed that the attentuation due to the deposit was to a close approximation the same as it would have been had the particulate matter been dispersed in a column of air of the same cross-sectional area.

Meteorological data included measurements of visual range, temperature and humidity from which average values during the sampling periods could be calculated.

COLLECTION OF AIRBORNE PARTICULATES

Particulate samples were collected at C.D.E.E. Porton by means of millipore filters at 20 ft above ground level. Two samples were taken simultaneously, one at 7·55 l/min for the transmission measurements and the other at 0·95 l/min for microscopic examination of the particles. The sampling rates were maintained constant by means of critical orifices. It is believed that the millipore filters gave truly representative samples of the particulates for there were negligible orifice losses.

Sampling was carried out for as long a period as possible each day over a period of several months, but not while there were any obvious fog or mist particles in the air, the commencement of sampling then being delayed until the fog or mist had disappeared. No samples were taken when there was continuous rain, although the sampling units were protected by a cover so that they were not spoilt by an occasional shower. Care was also taken to avoid sampling when smoke was being blown directly from a chimney in the neighbourhood towards the site.

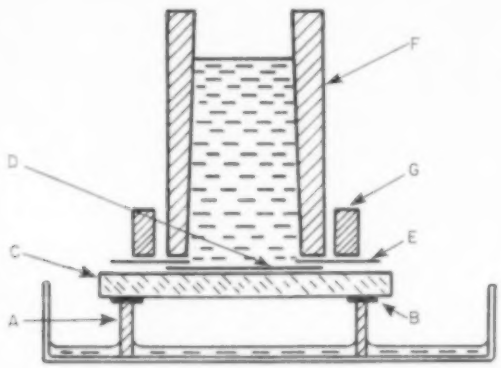


FIG. 1. Apparatus for transfer of deposit from millipore filter to sodium chloride disc.

At the end of the sampling time the millipore samples were removed and weighed, care being taken not to disturb the deposit of dust collected or to damage the filter.

The heavier sample was then transferred from the millipore filter to the surface of a sodium chloride disc without disturbing the relative positions of the particles, by a technique which is illustrated in FIG. 1. The sodium chloride disc, C, rested upon a brass ring, A, standing in a petrie dish, a thin rubber ring, B, being placed between them to prevent damage by scratching. The millipore filter was carefully held near the edge with a pair of tweezers and a small quantity of dry absolute alcohol was then put on the opposite side to the deposit, sufficient being applied to make the filter thoroughly wet, without dripping. This made the filter material slightly soft and flexible. It was then carefully lowered on to the sodium chloride disc, with the deposit on the under surface, care being taken to avoid trapping any air bubbles. A ring of No. 1 Whatman filter paper, E, was placed upon the filter, D, the centre hole corresponding to the area of the dust deposit. A few drops of alcohol were placed on the filter paper until it also was thoroughly wet and lying flat with no air bubbles. After that the short heavy brass

VOL. 5 1961

cylinder, F, was placed on the filter paper and very gently pressed down. Another short brass ring, G, was placed outside. The cylinder, F, was then very slowly filled with acetone which had been thoroughly dried by distillation over calcium chloride and the whole was covered over with a large inverted beaker to prevent evaporation which would have caused local cooling and condensation of water vapour on the sodium chloride disc. The acetone leaked out very slowly through the filter paper carrying with it the soluble millipore film. When about half the liquid had leaked away the cylinder was filled up again by careful addition of more acetone. The total time for all the liquid to leak away varied from about 20 min to 1 hr. In this way the undisturbed deposit was left upon the surface of the sodium chloride disc. The reason for the initial use of alcohol was that it prevented the formation of bubbles under the millipore filter and also if acetone was put directly on to the dry filter it would have immediately wrinkled up instead of lying flat upon the disc.

It is possible that some particles were lost during the process, but the number is believed to be relatively small since they were on the underside of the filter and would tend to stick to the surface of the disc with which they were in contact. It is improbable that they would be removed from this surface since the rate of flow of the liquid was extremely small. If an appreciable portion of the particles had been carried away by the liquid this would have been indicated by their reappearance at the edge of the Whatman filter paper and this did not occur. Soluble particles would of course have been removed completely. Here again the loss is not believed to be appreciable. Several tests were carried out in which, while a deposit was being observed under the microscope, water was allowed to seep along the filter and as it crossed the field of view none of the visible particles were seen to disappear. The losses, if any, must have been in the submicron range and these would have only a very minor effect on the infra-red transmission. It was not possible to carry out a similar experiment with alcohol or acetone but the amount of pollutant soluble in these liquids is normally very small.

The infra-red transmissions of the deposits were determined by means of a single beam, double monochromator of a design similar to one developed by A. JACKSON and L. A. SAYCE at the National Physical Laboratory (unpublished). It employed a sodium chloride prism, a "blazed" replica grating and a Golay infra-red detector. The transmission was measured by first obtaining a spectrogram with the clean sodium chloride disc inserted and then another with the deposit of pollutant on its surface. From six blank experiments, that is using new millipore filters with no deposits and carrying out the procedure exactly as above, it was found that a zero correction of 4 per cent to all readings was necessary. This was probably due to residual traces of the millipore filter material remaining on the disc.

The angle of the cone of the incident beam in the spectrometer at the position of the sample was about 10°, but according to GUMPRECHT and SLIEPCEVITCH (1953) and also LOTHIAN and CHAPPELL (1951), in view of the fact that the angle of the cone at the detector was very small, errors introduced by this were negligible.

RESULTS AND DISCUSSION

The period of investigation was from 27th June to 4th November, 1958. The values used for the visual range, temperature and relative humidity were averages for the periods of sampling, these being usually from about 9 a.m. to 3.30 p.m. G.M.T.

In FIG. 2 the visual range is plotted against concentration of the pollutant. A scale showing the corresponding attenuation coefficients is also shown. The wide scatter in the experimental points is probably mainly due to the variable composition and particle size distribution of the pollutant, although there is an indication that an

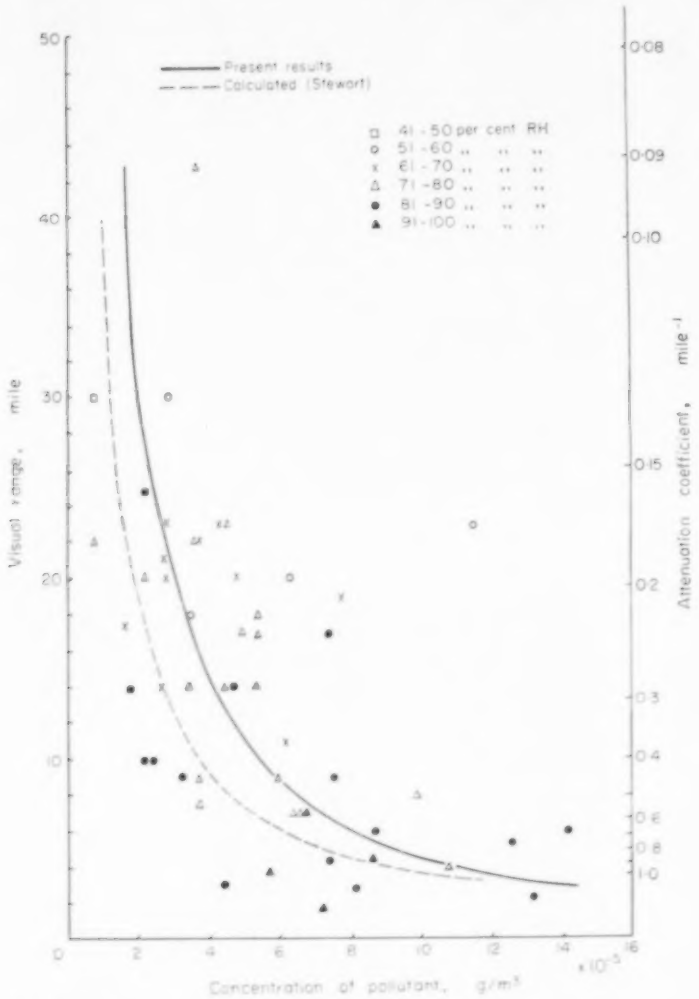

Present results
Calculated (Stewart)
41 - 50 per cent RH
51 - 60 " " "
61 - 70 " " "
71 - 80 " " "
81 - 90 " " "
91 - 100 " " "
Visual range, mile
Attenuation coefficient, mile⁻¹
Concentration of pollutant, g/m³
x 10⁻⁵

FIG. 2. Relationship between visual range and concentration of pollutant.

increase in humidity tends to decrease the visual range for a given concentration. The dotted curve is plotted on the basis of $\sigma = 6{\cdot}5w$ Km^{-1} according to STEWART (1956). This curve indicates a lower visual range for a given concentration than the full curve in FIG. 2, but the data from which it was derived were obtained in London whereas the present results were obtained in a location remote from an industrial area.

VOL. 5 1961

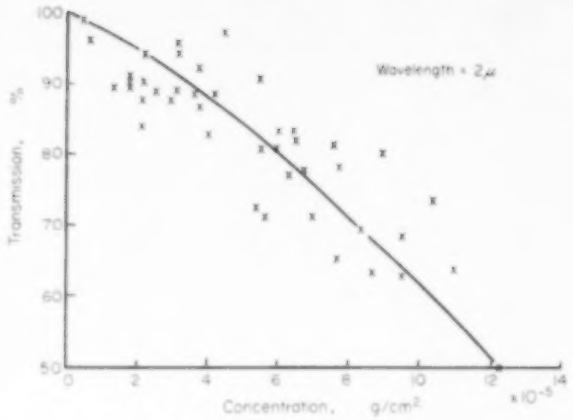


FIG. 3.

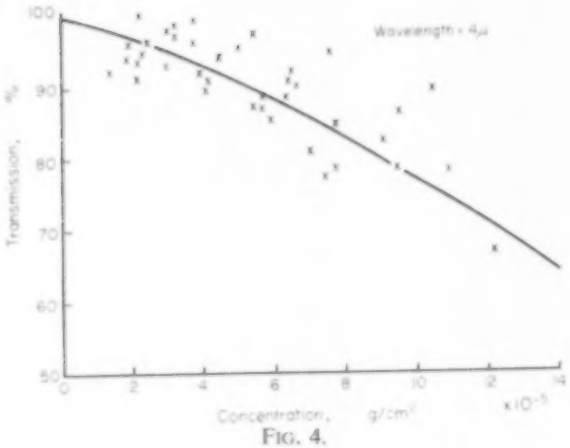


FIG. 4.

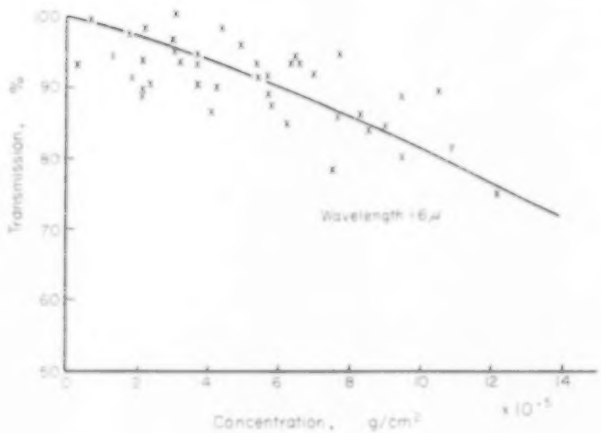


FIG. 5.

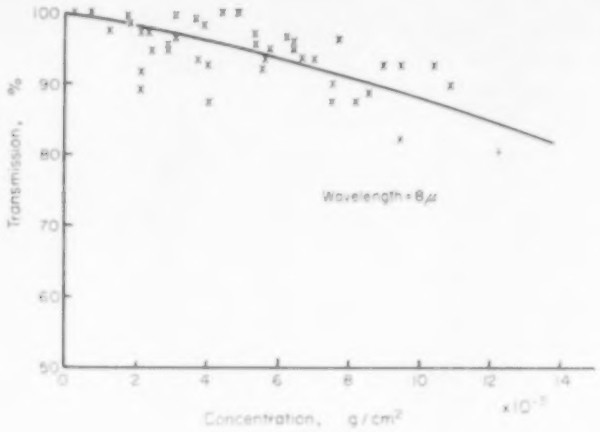


FIG. 6.

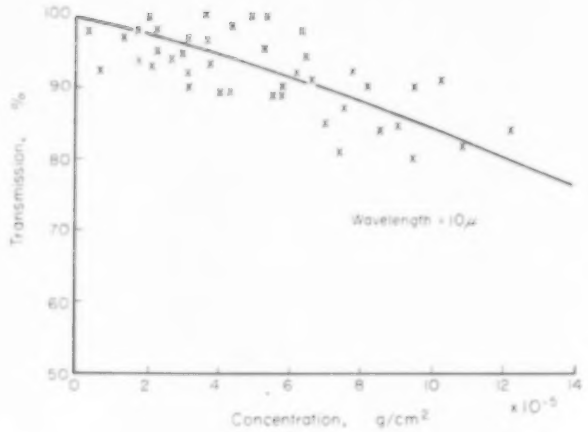


FIG. 7.

FIGS. 3 to 7. Relationship between percentage transmission of radiation and concentration of pollutant.

The samples taken for microscopic examination were not subjected to accurate particle size analysis because a large amount of the material collected was below the range of the light microscope and the particles were of very irregular shape; notes were, however, made on each sample, as observed under the microscope, with a 4 mm objective and a magnification of $\times 630$. It was found that all samples had a background of very small particles below 1μ, the majority being below $0{\cdot}6\mu$. Irregularly shaped particles up to about 5μ and occasionally, up to 15μ were also present. These varied in colour, being usually yellowish, transparent or black and it is probable that they consisted mainly of terrestrial dust. Samples from experiments covering the period July 4th–31st contained occasional "sausage" shaped particles which were probably

VOL. 5 1961

of organic origin. On most samples there were feathery chains or aggregates believed to be from smoke. Some samples also included opaque black spheres in the range 1·3μ to 1·7μ; more rarely yellow, transparent or black spheres up to 5μ were observed.

The results obtained from the infra-red transmission measurements are plotted in FIGS. 3 to 7 and show a general increase in the transmission with increasing wavelength as would be expected from classical Mie theory. These graphs were drawn subjectively by eye and although they are shown as curves, it is possible, that they should be straight lines, that is the attenuation is proportional to the concentration. It was, however, observed that the mean size of the atmospheric particles was larger when the concentration of the pollutant increased, in which case these graphs would be of the shape shown, since the attenuation increases rapidly as the particle size approaches the wavelength of the radiation.

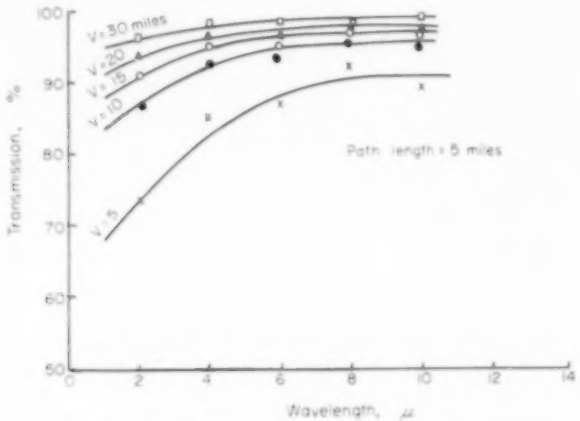


FIG. 8. Relationship between percentage transmission of radiation and wavelength.

When the visual range is known an approximate value of the concentration of pollutant can be obtained from the curve of FIG. 2. From this concentration the amount of pollution in a given path length can be calculated and hence the percentage transmission at a number of wavelengths can be read off from the graphs (FIGS. 3 to 7) of the experimental determinations of infra-red transmission. The percentage transmission can thus be plotted against wavelength for different visual ranges by making use of FIG. 2 and a series of curves can be derived as shown for example in FIG. 8 for a path length of 5 miles. This is the length corresponding approximately to the product of the mean linear sampling speed and the duration of sampling.

On account of this increase in particle size with concentration resulting in FIGS. 3 to 7 not showing a linear relationship, it is more accurate to use FIG. 8 to calculate the transmission for other path lengths than to use the method described above. It is within the limit of the experimental error to assume that the attenuation is proportional to the path length, if the transmission is above about 70 per cent, instead of using the exponential law.

It must be realized, however, that these curves indicate the transmission through

the solid atmospheric pollutant alone and to determine the actual transmission over a given path length, the transmission of the water vapour and carbon dioxide must also be taken into account. The visual range estimated from these curves would indicate a higher value than that observed.

The results are applicable strictly to the transmission of a collimated beam in the absence of multiple scattering. SHEPPARD (1958) suggests that as a rough rule it is safe to assume single scattering only if the transmission of the direct beam is not reduced to less than 0·9 by passage through the aerosol. If the transmission is between 0·74 and 0·90 there is some effect of secondary scattering, but multiple scattering prevails if it is less than 0·74. It would appear therefore that one is justified in assuming that to a first approximation, when the transmission is above about 74 per cent, the attenuation is proportional to the product of the concentration and the path length, i.e. to the number of particles in the path. It also supports the belief that the attenuation due to the deposit was the same as if the particulate matter were dispersed in a column of air of the same cross-sectional area.

Errors would have arisen if the particles on the filter overlapped to any great extent, in which case the curves of FIGS. 3 to 7 would have been concave upwards.

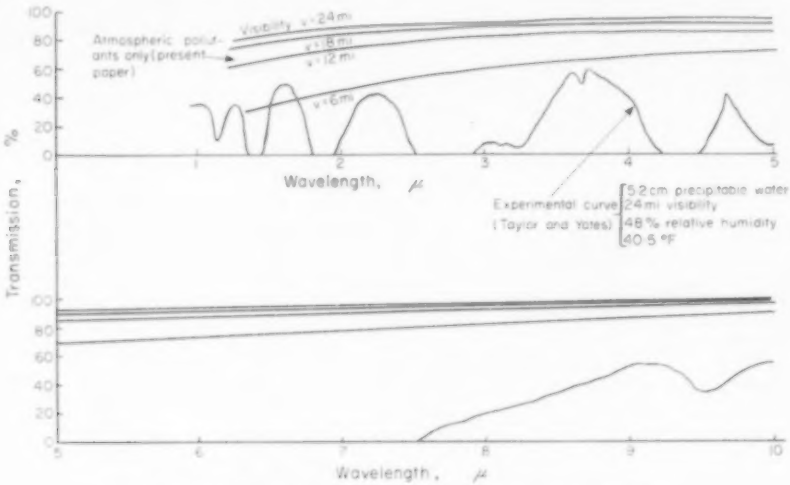


FIG. 9. Atmospheric transmission over a 10·1 mile path.

TRANSMISSION VALUES

To illustrate the implications of the results, a series of curves has been drawn in FIG. 9 relating transmission by atmospheric pollution to wavelength over a 10·1 mile path for different visual ranges. These may be compared with the experimental curves of TAYLOR and YATES (1957) for water vapour, carbon dioxide and atmospheric pollution over the same path length and a visual range of 24 miles. It is evident that the effect of suspended solid pollution in the atmosphere on the transmission of a collimated beam in the range of wavelengths 1 to 10μ is less than that due to water vapour and carbon dioxide except when the visual range is relatively low. The values for the transmission may be used to calculate the overall transmission of a collimated beam

VOL. 5 1961

of infra-red radiation for a source of a given colour temperature. For instance for a source at 6000° K in an atmosphere with a 24 miles visual range, the percentage of the total infra-red energy transmitted over a 10·1 mile path length would be 23 per cent while for the pollutant alone it would be 87 per cent. For a 6 mile visual range the transmission by the pollutant would fall to 40 per cent.

Previous data for the transmission of atmospheric haze have been based chiefly on the experimental work of GEBBIE and others (1951). YATES (1951) has plotted curves for the transmission through haze after correcting for water vapour and carbon dioxide absorption, as described above. LOCKE and collaborators (1956) have put forward similar curves showing the percentage transmission per nautical mile as a function of wavelength of the radiation and visual range but for a water vapour content of 17 mm precipitable water. TABLE 1 above was calculated from data in this paper based on the same experiments. ELDER and STRONG (1953) have shown that their results agree fairly well with GEBBIE and others.

The present results show a considerably higher transmission than those quoted above, the transmission for a 5 mile path being approximately the same as that for a 1 mile path at the same visual range. The experiments of GEBBIE and others and of STRONG were all carried out across coastal water whereas those reported here were carried out inland. It is therefore concluded that the higher transmissions found in this work are due to the atmospheric pollutant having a different composition and particle size range from the suspended matter over coastal waters.

Acknowledgements—I should like to express my thanks to Mrs. P. C. ERRINGTON and Mrs. M H. HARVEY who carried out the experimental work and most of the calculation of the results and to B.r. H. L. GREEN for helpful suggestions during the course of the work and for criticism of the manuscript. The meteorological data were supplied by the Meteorology Section at this Establishment. This paper is based on an internal report and is Crown copyright. It is reproduced with the permission of the Controller, H.M.S.O.

REFERENCES

DEIRMENDJIAN D. (1960) Atmospheric Extinction of Infra-red Radiation. *Quart. J. Roy. Met. Soc.* **86,** 371.

ELDER T. and STRONG J. (1953) The Infra-red Transmission of Atmospheric Windows. *J. Franklin Inst.* **255,** 189.

ELSASSER W. H. (1942) Heat Transfer by Infra-red Radiation in the Atmosphere. Harvard Met. Series No. 6. Harvard Univ.

GEBBIE H. A., HARDING R. W., HILSUM C., PRYCE A. W. and ROBERTS Y. (1951) Atmospheric Transmission in the 1 to 14μ Region. *Proc. Roy. Soc.* A, **206,** 87.

GUMPRECHT R. O. and SLIEPCEVITCH C. M. (1953) Scattering of Light by Large Spherical Particles. *J. Phys. Chem.* **57,** 92.

HULBERT E. O. (1954) Atmospheric Transmission of Infra-red Radiation. Naval Res. Lab. Report H. 2348.

KOSCHMIEDER H. (1924) Theorie der horizontalen Sichtweite. *Beitr. Phys. freien Atm.* **12,** 33.

LARMORE L. (1956) Transmissions of Infra-red Radiation through the Atmosphere. *Proc. Infra-red Symposium,* **1,** 14.

LOCKE A. S. and COLLABORATORS (1956) *Principles of Guided Missile Design: Guidance.* Vol. **1,** p. 161. Macmillan, London.

LOTHIAN G. F. and CHAPPELL F. P. (1951) The Transmission of Light through Suspensions. *J. Appl. Chem.* **1**, 475.

ROACH W. T. and GOODY R. M. (1958) Absorption and Emmision in the Atmospheric Window from 770 to 1250 cm^{-1}. *Quart. J. Roy. Met. Soc.* **84**, 319.

SHEPPARD P. A. (1958) The Effect of Pollution on Radiation in the Atmosphere. *Int. J. Air Poll.* **1**, 31.

STEFFENS G. (1956) *Air Pollution Handbook*, pp. 6–23. McGraw-Hill, New York.

STEWART K. H. (1956) Unpublished Work.

TAYLOR J. H. and YATES H. W. (1957) Atmospheric Transmission in the Infra-red. *J. Opt. Soc. Amer.* **47**, 223.

YATES H. W. (1951) Total Transmission of the Atmosphere in the Near Infra-red. Naval Res. Lab. Report 3858.

VOL.
5
1961

Int. J. Air Wat. Poll. Pergamon Press 1961. Vol. 5, No. 1, pp. 46–55. Printed in Great Britain.

THE EFFECT OF SUSPENDED MINERAL SOLIDS ON THE SURVIVAL OF TROUT

D. W. M. HERBERT and J. C. MERKENS

Water Pollution Research Laboratory, Stevenage, Herts.

(*Received* 17 *March* 1961)

INTRODUCTION

FINELY divided inert solid material of mineral origin in suspension in a river water may adversely affect fish in a number of ways. If it settles to form deposits it may silt up spawning beds and fill crevices in which invertebrates eaten by fish would hide; it may cut down the penetration of light, reducing the productivity of the river so that food for fish is less abundant, and also fish may be less easily able to hunt for food because visibility will be restricted. Effects of this kind are, however, not dealt with in this paper, which is concerned only with the effects which suspended solids may have more directly upon fish.

It seems to be generally accepted that for suspended solids to have a direct effect on fish, the concentration must be very high. WALLEN (1951) investigated the effect of erosion silt (mostly montmorillonite clay) on a number of species of North American fish and concluded that most endured a week in water in which the solids concentration was raised intermittently by hand stirring to 100,000 p.p.m., but finally died when the concentrations were raised to 175,000–225,000 p.p.m., these lethal turbidities causing death within 2 hr of exposure. GRIFFIN (1938) reported tests of 3–4 weeks' duration in which salmon and trout fingerlings withstood silt concentrations of 300–750 p.p.m. which were increased each day for a while when the tanks were stirred by hand to concentrations of 2300–6500 p.p.m. COLE (1935) found that a 2 per cent (20,000 p.p.m.) suspension of wood fibres was not harmful to healthy fish, although he says that it undoubtedly hastened the death of unhealthy or moribund individuals.

From this there is no doubt that many species of fresh-water fish can withstand extremely high concentrations of suspended solids for short periods, but this does not mean that much lower concentrations are harmless to fish which remain in contact with them for a very long time. It must be unusual for suspended-solids concentrations of several per cent to be maintained in rivers for long, but a number of industrial processes discharge wastes which maintain the suspended-solids concentration in streams at lower concentrations more or less indefinitely, and there is some uncertainty as to how harmful lower concentrations of solids are. Average values of about 100 p.p.m. have been reported for surface waters containing trout and other fish (WARD 1938), and SCHNEDEBERGER and JEWEL (1928) found that a decrease in turbidity down to an average of 100 p.p.m. was accompanied by an increase in fish production in ponds, so it is possible that suspended solids are harmless at concentrations less than about 100 p.p.m. The possibility that concentrations of suspended solids lower than 1000 p.p.m. may exert some directly harmful effect on fish has received relatively little attention, however, and this aspect of the problem is the subject of the investigations described in this paper.

METHODS AND MATERIAL

All the fish used were rainbow trout (*Salmo gairdnerii* Richardson) which were between about 9 and 18 months old at the start of an experiment. For each experiment fish of the same age were taken from one pond at a trout farm and allocated at random to the test concentrations; ten fish were used in each test. The dilution water was hard (320 p.p.m. as $CaCO_3$) and was pumped without treatment from a borehole; before use it was thoroughly aerated and brought near to room temperature.

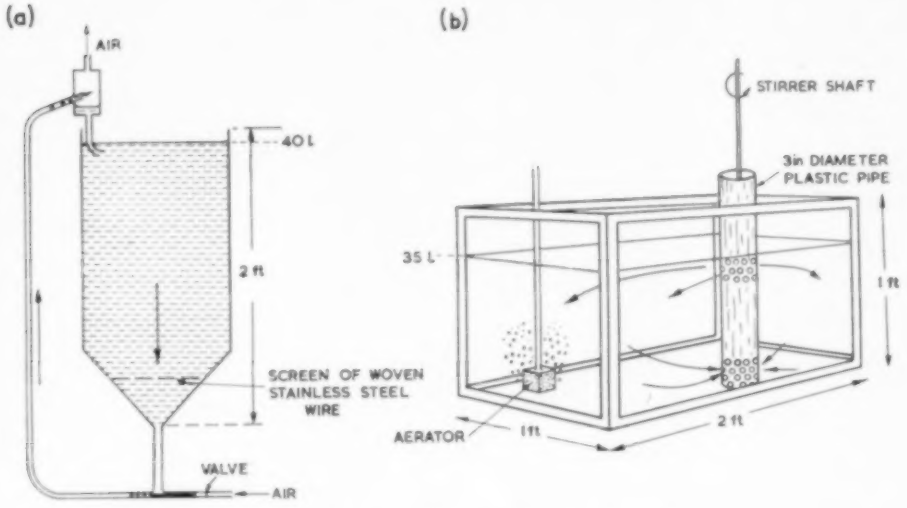


FIG. 1. Aquaria for maintaining solids in suspension.

The problem in designing suitable apparatus for the experiments was to prevent the added solids settling from the water without providing so much turbulence that the test tanks became unsuitable for the fish to live in for long periods. One type of apparatus (FIG. 1(a)) is a circular vessel, cylindrical at its upper part and conical at the base. The suspension is continually drawn from the bottom of the tank and returned to the top by an airlift, and is made to re-enter the tank tangentially to impart a gentle circular motion to the water, which helps to prevent solid matter accumulating on the walls of the base cone. With this apparatus no attempt is made to prevent solids settling out, but any which do are promptly returned to the surface. Although one series of tests (Experiment A) was made with these tanks, they were not very suitable for keeping trout in for long periods. The fish tended to swim round the tanks, frequently bringing their caudal fins into contact with the wall, and also they often settled on the wire mesh at the bottom. This resulted in a high proportion of the fish developing sores on those parts of the body which were frequently abraded.

The test vessel which was eventually designed, and was used in all the experiments reported here except Experiment A, is illustrated in FIG. 1(b). Each vessel consists of an ordinary glass aquarium measuring 2 ft × 1 ft × 1 ft; it contains 35 litres of water or suspension. Standing on the base and equidistant from either end is a length of

VOL. 5 1961

plastic pipe of 3-in. diameter with holes, about ¼-in. diameter, bored through its wall near the base and just below the waterline. Half-way between the sets of holes a stainless steel propeller revolves at 900–1000 r.p.m. to draw water in through the lower holes and expel it through the higher set. This device circulates water at a rate of about 35 litres/min. Some additional turbulence is introduced by aerating the water or suspension through a small diffuser at the end of each tank.

Table 1. Particle-size distributions of kaolin and diatomaceous earth used in experiments with fish

(a) By sedimentation method

Kaolin		Diatomaceous earth	
Particle size (microns)	Per cent (by weight)	Particle size (microns)	Per cent (by weight)
Coarser than 5	1	Coarser than 10	5
5–1	14	10–6	8
1–0·5	30	6–2	37
0·5–0·13	50	2–1	26
Finer than 0·13	5	Finer than 1	24
Median 0·46 μ		Median 2·2 μ	

(b) By optical method

Kaolin		Diatomaceous earth	
Particle size (microns)	Per cent (by weight)	Particle size (microns)	Per cent (by weight)
Coarser than 9·4	4	Coarser than 37·5	22
9·4–4·7	18	37·5–18·8	26
4·7–2·3	48	18·8–9·4	20
	30	9·4–4·7	23
Finer than 2·3		4·7–2·3	8
		Finer than 2·3	1
Median 3 μ		Median 17·5 μ	

All fish were fed, nearly to repletion, once each day on a mixture of minced bullock's heart and liver. Shortly afterwards they were removed from the aquaria, which were then dismantled, thoroughly cleaned, then reassembled and refilled with the appropriate concentration of suspended matter; the fish were then returned. In preliminary work it was found that fish often lost condition if they were transferred from stock directly to aquaria being stirred at the full rate, so before any solids were added the fish were acclimatized to the experimental conditions for one or two weeks, the speed of stirring being gradually increased to the final value. In some preliminary experiments with other kinds of test vessel, outbreaks of fin-rot occurred among the fish. As

a precaution against this a little terramycin was added each day to the water as recommended by WALLEN, GREER and LASATER (1957); it did not completely prevent this disease however.

The solids employed were commercial grades of kaolin and diatomaceous earth. Particle-size distributions of these powders, determined by a sedimentation method,

TABLE 2. SUMMARY OF CONDITIONS PREVAILING IN TEST VESSELS
(Figures in parentheses are graphical estimates of standard deviation)

Experiment and date started	Apparatus used	Solid material	Concentration of solids added (p.p.m.)	Mean concentration of solids suspension by analysis (p.p.m.)
A 5th Jan. 1959	Circular tanks with conical base	Kaolin	0 30 90 270 810	5 (2) 21 (2) 88 (12) 264 (30) 780 (30)
B 16th Feb. 1959	Stirred rectangular aquaria	Kaolin	0 30 90 270 810	7 (4) 27 (9) 76 (11) 240 (30) 717 (50)
C 16th Mar. 1959	Stirred rectangular aquaria	Kaolin	0 30 90 270 810	1 (1) 22 (7) 72 (24) 201 (24) 657 (30)
D 19th May 1959	Stirred rectangular aquaria	Diatomaceous earth	0 30 270 810	6 (4) 25 (9) 245 (20) 740 (35)
E 19th May 1959	Stirred rectangular aquaria	Diatomaceous earth	0 30 90 270 810	10 (5) 22 (8) 93 (17) 240 (25) 730 (50)
F 18th Aug. 1959	Stirred rectangular aquaria	Diatomaceous earth	0 30 90 270 810	9·5 (2) 24 (5) 84 (10) 217 (20) 714 (30)

were given by the suppliers and are summarized in TABLE 1(a). Sizes are expressed as equivalent diameters in microns according to Stoke's law of settling. Mr. HAMILTON, of the National Coal Board's Mining Research Establishment, kindly arranged for size distributions to be determined by an optical method described in HAMILTON and KNIGHT (1958), which is a modification of a method given by FAIRS (1951). These results are in TABLE 1(b).

VOL. 5 1961

The optical method gives distinctly larger measurements for both materials. Differences of the order found are not uncommon when optical and sedimentation methods are compared because the methods measure different properties of the material. By the optical method the areas of the particles are measured after they have settled on a

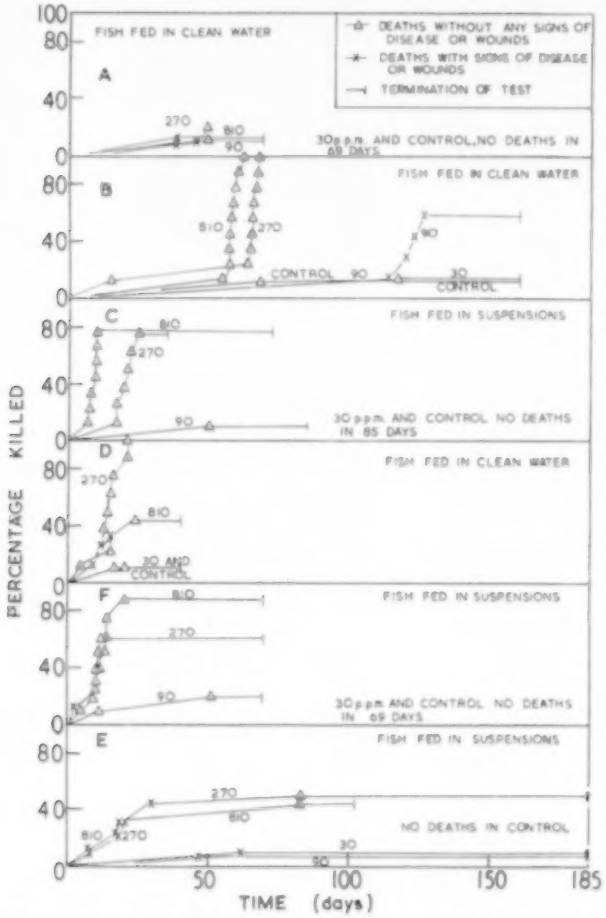


FIG. 2. Survival of rainbow trout in suspensions of kaolin (A-C) and diatomaceous earth (D-F).
Experiment A in conical tanks.
Experiments B-F in stirred rectangular aquaria.
Concentration of suspended solids in p.p.m. shown against each curve.

microscope slide, and because kaolin consists of flat plate-like particles and many of the diatomaceous earth particles are also flat, they will tend to settle on the surface of greatest area. Also, particles smaller than about 1–2 microns in diameter are not measured by the optical technique. In sedimentation methods size is expressed as the diameter of a sphere of material having the same density and settling velocity, but flat

particles would sink more slowly than spheres. All these factors will make sizes measured by the optical technique larger than those obtained by sedimentation methods.

All experiments were made at room temperature and analyses of the conditions prevailing in the test vessels were made at intervals, usually of one week but sometimes less frequently. The results of the suspended-solids analyses are summarized in TABLE 2. Dissolved-oxygen concentrations were always near the air-saturation value and pH values were in the region of 8·0–8·4. Temperatures varied with room temperature over the range 13–20° C.

RESULTS

Effect of suspended solids on survival

Data for the survival of the fish are given in FIG. 2. All deaths are included except for a few accidental deaths due to known causes (e.g. damage in transfer during aquarium cleaning). Some of the fish dying in the suspensions of solid matter showed obvious signs of disease, usually fin-rot, or diseased patches on the tail and flanks thought to be infected wounds gained through fighting other fish in the aquaria. Such deaths are distinguished in FIG. 2 from those occurring with no external symptoms of disease, and were in a majority only in Experiment F.

In all experiments the mortality among the controls was negligible, there being only two deaths among the 60 control fish put on test. Hence, any mortality in other tests appreciably in excess of this control level can be attributed with fair confidence to some effect of the added solids on the fish. From the data available there appears to be no great difference in lethal effect between the kaolin and the diatomaceous earth. Deaths in the aquaria to which 30 p.p.m. of either solid were added were at almost the same low level as among the controls, so there is no evidence that this concentration caused any harm. Except in Experiment B, there were few deaths in concentrations of 90 p.p.m., but since in most experiments mortalities at this concentration were slightly higher than in 30 p.p.m. or in the control tests, 90 p.p.m. appears to have had some adverse effect. In the higher concentrations the mortalities varied considerably from one test to another, but there were always some deaths, usually in excess of 50 per cent, and always more than in 0, 30 or 90 p.p.m. Under the test conditions, therefore, the kaolin and the diatomaceous earth were harmful to rainbow trout in concentrations of 270 p.p.m. and higher.

It was noticed that trout which had been kept for some weeks in the higher concentrations of solids were extremely pale, and almost white in colour.

Effect of suspended solids on the growth of trout

Some observations were made to see whether fish kept in the suspensions grew as well as those in the control aquaria, but since some proportion of the fish kept in the higher concentrations of solids died in the course of the tests, comparisons could only be made of the growth of the survivors. In Experiment F the majority of fish survived some months in all the concentrations except 810 p.p.m. All fish were weighed and measured on the first day of the experiment and the survivors again after about $4\frac{1}{2}$ months (FIG. 3). Analysis of variance showed no statistically significant difference between either the lengths or the weights of the fish exposed to the different concentrations of diatomaceous earth. This conclusion is supported by casual observations

VOL. 5 1961

made during other experiments; surviving fish seemed to grow just as well as the controls even in the higher concentrations.

Pathological changes in the gills

The gills of some trout which died in the experiments were dissected out and prepared for histological examination, but it was soon found that even if the fish had been dead for only a few hours before the gills were put into fixative, post-mortem changes had occurred and the detailed structure of the gill epithelium could not be made out. To obtain good preparations it was necessary to fix the gills soon after death, and since fish in the higher concentrations of solids could hardly be seen, and most deaths occurred outside working hours, only the gills of a small number of fish dying in the experiments were suitable for examination.

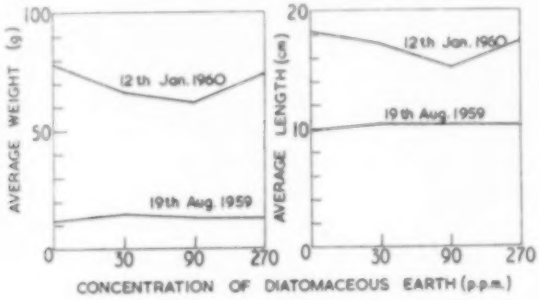
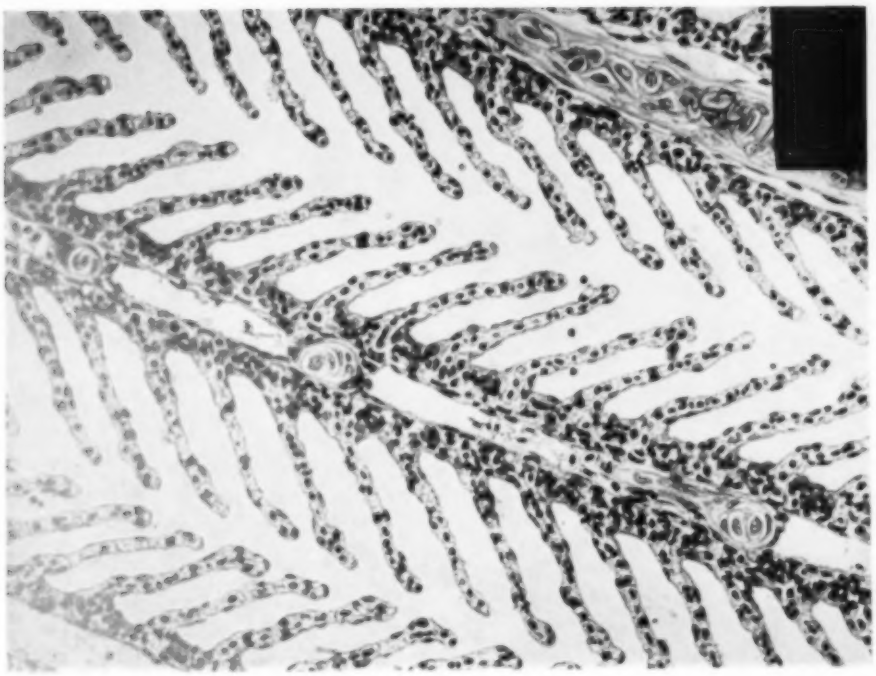


FIG. 3. Growth of rainbow trout in suspensions of diatomaceous earth.

Gills from three fish dying in 270 p.p.m. china clay in Experiment C and one fish dying in 810 p.p.m. diatomaceous earth in Experiment E were available for examination. In all of them a similar pattern of histological change was observed; the cells of the respiratory epithelium were much thicker than in normal gills, and in places adjacent lamellae were fused, frequently at the tips. Photomicrographs of an affected gill and of a normal gill are shown in FIG. 4. The two fish surviving in 270 and 810 p.p.m. china clay until the end of Experiment C were also affected, but the gills of the surviving fish in 810 p.p.m. diatomaceous earth (Experiment E) were apparently normal.

At the outset of all tests it was intended to continue them for at least three months, but several were stopped before this because some accident occurred. Experiment C was ended prematurely by accident, and in consequence no gills from the lower concentrations of china clay or the control aquaria in this experiment could be compared with the affected gills from 270 p.p.m. However, some gills were available from Experiment B which had been started before Experiment C and was continued for some months after it had ended. Four fish were examined at the end of the test in 30 p.p.m. and two fish at the end of the test in 90 p.p.m. All had normal gills although they had been in the suspensions for more than 5 months. From Experiment E in the diatomaceous earth, three control fish and two fish from 30 p.p.m. were selected for examination after more than 2 months in the suspensions and all these had normal



VOL.
5
1961

Fig. 4. Photomicrographs of sections of gills of rainbow trout:
(a) from normal trout

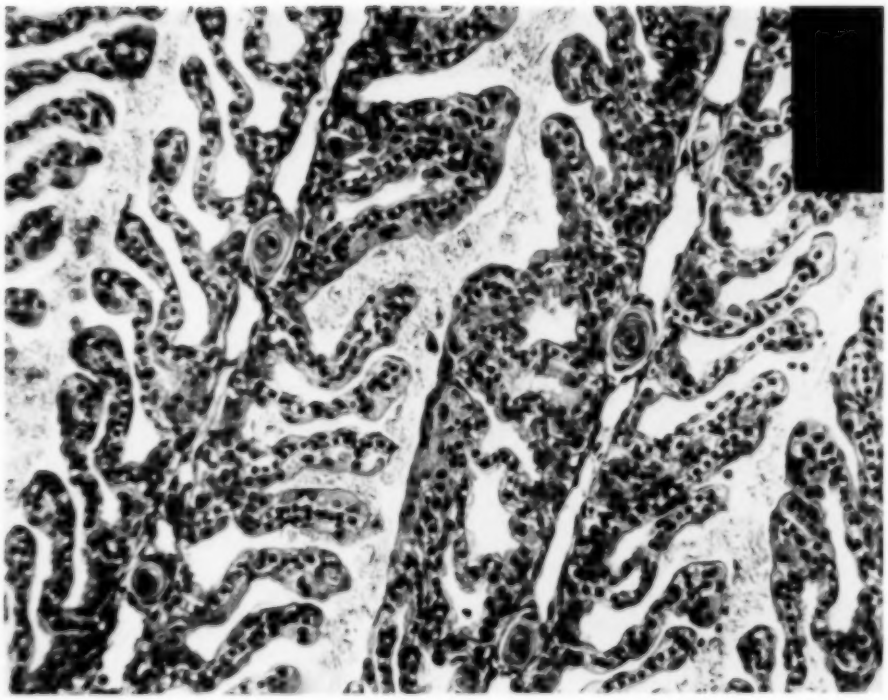


Fig. 4

(b) from trout which died after 13 days in a suspension of 810 p.p.m. diatomaceous earth. Note thickening of epithelial cells and fusion of adjacent lamellae.

gills. This suggests that the changes in histological structure were induced by the presence of the higher concentrations of solids in suspension.

In the hope of discovering more about the matter, another experiment (Experiment G) was started in which some rainbow trout were kept in suspensions of 270 p.p.m. diatomaceous earth and others in clean water. Individuals were removed at intervals and killed so that their gills could be examined. After 148 days, half the fish in the suspensions were transferred to 1000 p.p.m. diatomaceous earth for the remainder of the experiment, which continued for nearly 9 months. The results are summarized in TABLE 3. In most cases about 6–10 sections were examined from each of two gill arches from each fish. A set of gills is classed as affected in TABLE 3 only if it appeared definitely to be so by comparison with the gills of control fish.

TABLE 3. EFFECT OF DIATOMACEOUS EARTH ON THE GILLS OF RAINBOW TROUT

Days from start of experiment	Concentration of diatomaceous earth (p.p.m.)		
	0	270	270 for 148 days followed by 1000 p.p.m.
0	N	N	
37		3N	
78	N	N+2SA	
89		N	
91		N+MA	
100		SA	
101		N	
104		N+SA	
141	2N		
150		N	
171	N	N	N
196	N	N	SA
245	N	N	SA
265	N	2N	N+SA

N = Normal
SA = Slightly affected
MA = Moderately affected

The conclusion reached from TABLE 3 is that the thickening and fusion of the gill lamellae was associated with the presence of the suspended solids, although the condition seemed to take longer to manifest itself than it did in Experiments C and E. Furthermore, although some individuals were affected after about 11 weeks, others had perfectly normal gills after eight months or more in contact with the suspensions.

Suspended solids and the incidence of fin-rot

As has been stated, a number of fish in Experiments A to F died showing symptoms of the disease fin-rot; the caudal fin was usually most severely affected. While Experiment G was in progress the condition of this fin was recorded at intervals. An arbitrary scale was adopted as follows:

VOL. 5 1961

Grade	State of caudal fin
1	Normal.
2	Slightly ragged at edge.
3	Loss of tissue at edge, about ¼ of fin destroyed.
4	More than half of fin destroyed.
5	Fin completely gone.
6	Body wall infected near base of fin.

All the fish were at Grade 1 at the start of the experiment. When examined after 57 days the controls were still all at Grade 1, but the trout in 270 p.p.m. diatomaceous earth averaged Grade 2. After 121 days the controls were still all at Grade 1, but the fish in the suspensions averaged 2·9. There is no doubt that the trout in the suspensions suffered more from infected fins than the controls.

DISCUSSION

When the toxicity of a poison to fish is investigated, it is usual to find some definite and reasonably reproducible relation between period of survival and concentration. There was no such well-defined relation with rainbow trout in the suspensions investigated here, and this may be because chemically inert solids exert their harmful effect in a different way from soluble poisons.

Continual abrasion by the solid particles may have induced the thickened gills which were observed in some trout from the higher concentrations of solids. These were almost certainly less efficient than the normal organs; a fish suffering from this condition would be less viable than a normal individual and such a lesion might itself cause death if the thickening were severe enough. However, some individuals exposed to the suspensions for many months showed no sign of this condition, and the time needed to induce it in susceptible individuals seems to have been very variable; this may account to some extent for the variable mortalities occurring in the experiments.

FISH (1935) has published photomicrographs of trout gills which are very similar in appearance to those observed in the affected fish from the suspensions of solids. A similar condition was ascribed by DAVIS (1926 and 1927) to the irritation produced by a bacterium which grew on the gill surface but did not invade the tissue, but FISH was unable to find the causative organism in the cases he investigated, and concludes that more than one organism may be capable of producing the lesion. Possibly thickening and fusion is a general kind of response which can be evoked by more than one kind of stimulus, including irritation by inorganic solids. On the other hand, constant passage of solids over the gill epithelium may increase the chance of infection by a micro-organism.

In the latter case, the solids would have reduced the trouts' chances of survival by making them more susceptible to other adverse factors in their environment. This seems to have happened in those tests in which some fish suffered from fin-rot, which is said to be a bacterial disease; small wounds and abrasions which all fish, including controls, must have received from time to time possibly healed less readily when continually bathed with a suspension of hard particles, and so were more likely to become infected.

This suggests that some of the reduction in survival chances brought about by inert mineral solids in concentrations from 90 to 810 p.p.m. occurred because the solids potentiated other stresses in the environment which were not severe enough to produce more than a negligible mortality among the control fish and those in 30 p.p.m. Many of these stresses were not under experimental control, which may account for the variable mortalities in the higher concentrations. In aquaria the dominant stresses are likely to be those consequent upon overcrowding—fighting and a favourable environment for the spread of disease—while in a river these stresses may be less important than others. Consequently it cannot be said how far the results of this laboratory study are directly applicable to the survival of fish in rivers polluted with inert solid material, but a stress which reduces survival chances in one environment will probably reduce them to some extent in another, although not necessarily to the same degree.

Acknowledgements—Messrs. W. F. Ross and A. C. Wakeford assisted with the experimental work. This paper is published by permission of the Department of Scientific and Industrial Research.

REFERENCES

Cole A. E. (1935) Water pollution studies in Wisconsin. Effects of industrial (pulp and papermill) wastes on fish. *Sewage Wks J.* **7,** 280–302.

Davis H. S. (1926) A new gill disease of trout. *Trans. Amer. Fish. Soc.* **56,** 155-159.

Davis H. S. (1927) Further observations on the gill disease of trout. *Trans. Amer. Fish. Soc.* **57,** 210–212.

Fairs G. L. (1951) Developments in the technique of particle-size analysis by microscopical examination. *J. Roy. micr. Soc.* **71,** 209–222.

Fish F. F. (1935) A western type of bacterial gill disease. *Trans. Amer. Fish. Soc.* **65,** 85–87.

Griffin L. D. In Van Oosten J. (1945) Turbidity as a factor in the decline of Great Lake fishes, with special reference to Lake Erie. *Trans. Amer. Fish. Soc.* **75,** 281–322.

Hamilton R. J. and Knight G. (1958) *Some studies of dust-size distributions, and the relationship between dust formation and coal strength.* Proceedings of conference on mechanical properties of brittle materials. Butterworths Scientific Publications, London.

Schnedeberger E. and Jewel M. E. (1928) Factors affecting pond fish production. *Bull. Kans. For. Fish. Game Comm.*, No. 9, 14 pp.

Wallen I. E. (1951) The direct effect of turbidity on fishes. *Bull. Oklahoma Agric. Mech. College, Stillwater.* Oklahoma Arts and Science Studies. *Biol. Ser.* 2, **48,** No. 2.

Wallen I. E., Greer W. C. and Lasater R. (1957) Toxicity to *Gambusia affinis* of certain pure chemicals in turbid waters. *Sewage Industr. Wastes* **29,** 695–711.

VOL.
5
1961

Int. J. Air Wat. Poll. Pergamon Press 1961. Vol. 5, No. 1, pp. 56–74. Printed in Great Britain.

THE EFFECT OF CHINA-CLAY WASTES ON TROUT STREAMS

D. W. M. Herbert,* J. S. Alabaster,† M. C. Dart, and R. Lloyd*

(*Received* 17 *March* 1961)

INTRODUCTION

In some Cornish rivers suspended matter from china-clay workings is the only important polluting material, and such streams provide good sites for investigating the effect of chemically inert suspended solids on fisheries, since there are other unpolluted streams of similar size and character nearby. This paper describes a survey made during May 1960 to determine the status of the brown trout (*Salmo trutta* L.) in both clean and polluted parts of the Rivers Par, Fal, and Camel, and also in two unpolluted streams, the Luxulyan, which is a tributary of the Par, and the Tresillian. The concentrations of suspended matter, the numbers, size, and age of the trout, and the amount of food available to them in these streams were investigated.

DESCRIPTION OF AREA

The areas investigated and the sampling stations are shown in Figs. 1 and 2. The streams rise 500 to 800 ft above sea level, and first flow mainly through moorland where the channels are frequently overhung with scrub; the lower reaches flow through woodland and meadows. The stream beds generally consist of boulders, stones, and gravel, with very little finer material except for deposits of clay wastes in polluted stretches; there is little rooted vegetation. The widths of the streams range from 6 ft near the source to 15 to 20 ft at places lower down, and depths at the centres of the streams at the time of the survey varied from 6 in. to 4 ft.

China clay is formed by the natural decomposition of felspar or granite and is associated with particles of quartz, felspar, and mica. Much water is used to mine, transport, and refine china clay, and the large volumes discharged as wastes contain sand, mica, and clay in suspension, the proportions depending on the details of the industrial processes, and whether the effluent is treated before discharge.

PHYSICAL AND CHEMICAL PROPERTIES OF THE POLLUTED STREAMS

Sampling and methods of analysis

At most control stations the water was crystal clear, and the concentrations of suspended solids were not determined. Automatic vacuum samplers (Eden and Melbourne, 1959) were installed at the places shown in Figs. 1 and 2. Samples were taken at intervals of 6 hr. At Stations 11, 15, 16 and 20, samplers were operated from 28th April to 24th May (the period during which the biological surveys were made). At

*Water Pollution Research Laboratory, Stevenage.
†Ministry of Agriculture, Fisheries and Food, London.

Station 9 sampling was carried out from 28th April to 13th May, and at Station 13 from 13th May to 23rd May. At Station 25 on the River Camel, sampling by automatic sampler was not started until 7 weeks after the biological survey was made there, and was continued from 12th July until 1st August. Results from this automatic sampler showed similar concentrations of suspended solids to those of a few samples taken by hand at the time of the biological survey.

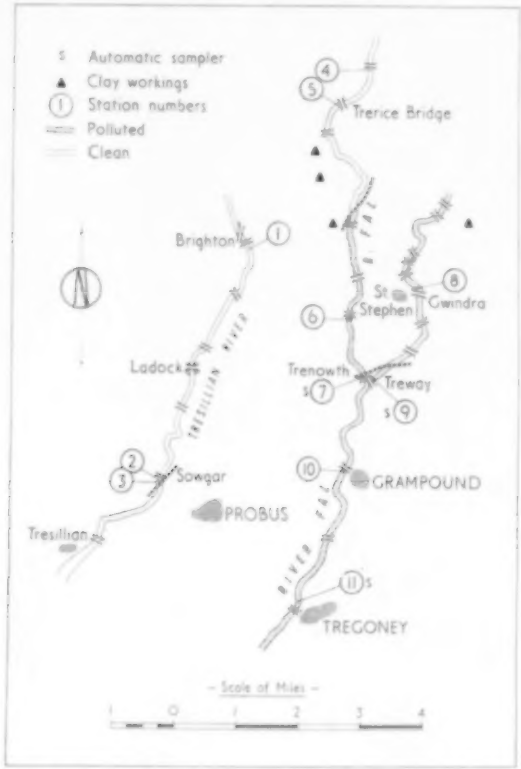


FIG. 1. Sampling stations on the Rivers Tresillian and Fal.

Each of the samples was analysed for total suspended solids by filtration through a prepared Gooch crucible (Ministry of Housing and Local Government, 1956). Particle-size distributions were determined by sieving and sedimentation as described in British Standard 1377 : 1948, Method A. To detect any gross variations in particle size occurring during the survey, at most sampling stations on the Par and Fal aliquots of the 6-hourly samples taken during the first fortnight were pooled and examined, and the results for each station compared with those from similar samples for the second fortnight. Samples of river water were centrifuged, and the supernatant analysed for dissolved mineral content. Carbonate, nitrate, chloride, phosphate, and sulphate were estimated by methods recommended by the Ministry of Housing and Local Government (1956). Sodium was determined volumetrically by the zinc uranyl

VOL. 5 1961

acetate method, and potassium by the cobalti-nitrate method (VOGEL, 1957). Calcium and magnesium were determined by complexing with E.D.T.A. (DIEHL, GOETZ and HACH, 1950; BETZ and NOLL, 1950). In addition, a few field determinations of dissolved oxygen were carried out by the modified Winkler method (Ministry of Housing and Local Government, 1956). The pH value and the conductivity were determined occasionally with portable meters.

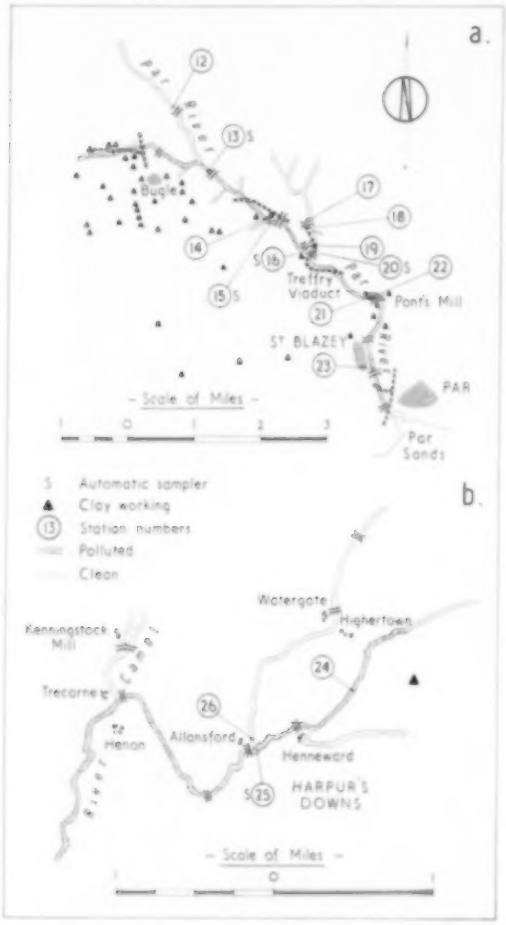


FIG. 2. Sampling stations on the Rivers Par (2a) and Camel (2b).

Results

The variation of suspended solids with time at two sampling stations is shown in FIG. 3. No regular daily variation is apparent from these graphs; the only regular features were the startling increase in suspended solids which occurred on Saturday evenings at Station 16 because a settlement tank was flushed out at this time, and a tendency for the solids concentrations at this Station to be rather lower on Sunday and

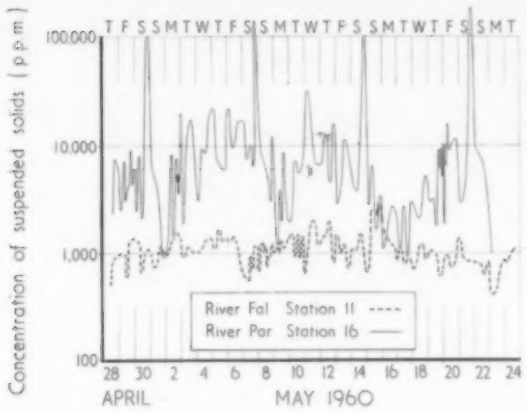

T F S S M T W T F S S M T W T F S S M T W T F S S M T
100,000
10,000
1,000
100
Concentration of suspended solids (p.p.m)
River Fal Station 11
River Par Station 16
28 30 2 4 6 8 10 12 14 16 18 20 22 24
APRIL
MAY 1960

FIG. 3. Fluctuations in suspended solids concentration in the Rivers Par and Fal.

VOL. 5 1961

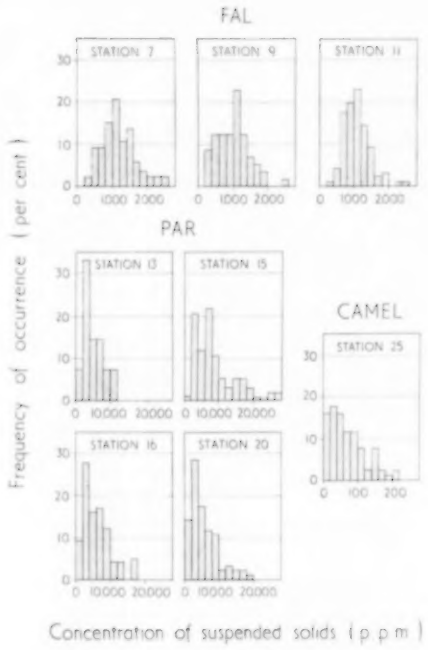


FIG. 4. Histograms showing frequency of occurrence of suspended solids at sampling stations where automatic samplers were in operation

early on Monday than on other days of the week. The concentration of suspended solids at Station 11 on the River Fal was generally less (median 1040 p.p.m.) than at Station 16 on the River Par (median 5100 p.p.m.) and was not subject to such great variation.

It is evident from the histograms (FIG. 4) that the distribution of the suspended solids concentrations was usually skew and analysis of the data showed that at most sampling stations the concentrations had an approximately logarithmic normal distribution. The median concentration is thus best estimated as the geometric mean of the concentrations; these and the standard deviations of log concentration were calculated for each station, and are given in TABLE 1.

TABLE 1. CONCENTRATION OF SUSPENDED SOLIDS IN SOME CORNISH STREAMS

River system	Station No.	Median concentration (p.p.m.)	Standard deviation of log concentration
Par	13	6040	0·44
,,	15	7470	0·34
,,	16	5100	0·38
,,	20	4740	0·41
Fal	7	1210	0·91
,,	9	934	0·24
,,	11	1040	0·16
Camel	25	58·6	0·41

On the polluted part of the River Fal the sampling points for suspended solids were selected to give fairly representative coverage, and from the geography of the area (FIG. 1) and the records of suspended solids concentration (TABLE 1) it appears that living creatures at all the polluted sampling stations must have been subject on average to much the same concentrations of suspended solids. The same is probably true of the River Par, for although no sampler was installed below Station 20, no substantial quantities of either clean or polluted water enter this river between Stations 20 and 23.

The distributions of particle size are shown in FIG. 5. The most frequently occurring size range was 6–60 microns, and there was no marked difference in particle-size distribution between the first and second fortnights of the survey, except possibly at Station 11.

The average values of the mineral analyses for the polluted stretches of the Par, Fal, and Camel river systems are presented in TABLE 2. All the waters were similar in character, except the small tributary of the Camel in which nitrate and chloride were much lower, probably because it flowed through sparsely populated country and received no sewage effluent.

Occasional determinations showed that dissolved-oxygen concentrations were always close to the air-saturation value, pH values were between 6·8 and 7·2, conductivities between 70 and 270 micromhos/cm³, and temperatures between 10 and 15° C.

FISH POPULATIONS

Methods

At each of the sampling points shown in FIGS. 1 and 2 a length of stream 50 to 100 yd long was chosen, giving a stream bed area of 1000 to 5000 ft². Stopnets of ½-in. mesh were put across the river at the upstream and downstream ends of the sampling area, the lower net having a narrow piece of netting attached to the bottom and held away from the upstream side of the stopnet to form a non-return flap (ELSON, 1960). The

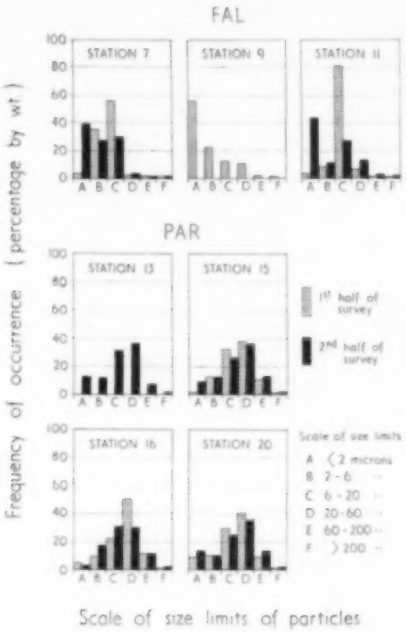


FIG. 5. Particle-size distributions of the solids in suspension at sampling stations where automatic samplers were in operation.

TABLE 2. AVERAGE MINERAL ANALYSES FOR SOME CORNISH STREAMS
(Concentrations in p.p.m.)

Radical	River system		
	Par	Fal	Camel
Calcium	5·4	7·7	7·1
Magnesium	4·7	6·0	5·2
Potassium	2·1	3·1	3·6
Sodium	14·4	20·9	10·3
Carbonate + Bicarbonate	35·4	34·0	26·9
Chloride	23·5	28·7	14·0
Nitrate	7·2	9·4	nil
Phosphate	0·2	0·8	0·3
Sulphate	33·1	46·1	28·4

VOL. 5 1961

area between the stopnets was then fished with electric stunning gear. The machine most often employed was a 2·5 kW D.C. generator. The cathode was a plate of expanded aluminium submerged in the stream, and the anode with which the fish were stunned was a shallow bowl of expanded aluminium with an area of 140 in.^{2} fixed to a bamboo handle. The usual fishing technique was to work downstream towards the lower stopnet with an anode voltage of 210 to 240. The operator stood in midstream and, after switching on the current, moved the electrode over the bed of the stream from the bank to the fast flowing centre and then switched the current off. By this means fish near the margins of the streams were attracted to the anode, stunned, and carried down in the main body of water towards the stopnet. With another technique, often used in clean streams, the fish were caught in a hand-net as soon as they were stunned. Every sampling area was fished at least twice. The majority of fish caught were trout; some eels, lampreys, miller's thumbs, and fry were present, but they often slipped through the stopnets and are not considered further here. After each run the trout caught were measured from the snout to the fork in the tail, and were then either marked by clipping not more than 2 fins and returned to the sampling area, or else killed and weighed. The stomachs, scales, and gills of many fish were kept for examination.

At control stations where the fish were fairly abundant, population estimates were usually made by a catch-effort method (LESLIE and DAVIS, 1939) in which fish which are caught are either not returned to the stream or are marked so that they can readily be distinguished if they are caught a second time. The techniques used to fish each station were standardized as far as possible so that each fishing run should exert a constant fishing effort, the numbers of unmarked fish caught during each run were plotted against the accumulated catch, a straight line was fitted to the points by eye, and the population estimated as the cumulative catch corresponding to the point where the straight line cut the abscissa.

If the captured fish are marked and returned, populations may also be estimated by a mark-recapture method (CHAPMAN, 1952). At three control stations the two methods were compared; the population estimates obtained agreed very well (TABLE 3), and because the theories of the catch-effort and mark-recapture methods involve different assumptions about the catchability of fish, the agreement between them suggests that both provided valid estimates of the populations.

TABLE 3. COMPARISON OF CATCH-EFFORT AND MARK-RECAPTURE METHODS OF ESTIMATING POPULATIONS. (FIGURES IN PARENTHESES REPRESENT 95% FIDUCIAL LIMITS OF ESTIMATES USING DELURY'S METHOD (1958))

River system	Station No.	Estimated population between stopnets	
		Catch-effort method	CHAPMAN'S method
Fal	4	28	26 (22–31)
Par	17	34	35 (27–50)
,,	18	25	26 (19–39)

Where populations are small—as they were at most polluted stations—the catch-effort method is not suitable, and since it was desired to kill and examine as many fish as possible from these places, the modified mark-recapture method was not very suitable either, because this involves marking and returning to the stream all the fish which are caught, with the risk that some may not be recaptured. To overcome these difficulties a method was devised by one of us (J. S. ALABASTER) in which trout (usually about 20) were taken from streams outside the survey area or from control stations. They were marked, and then put at random into the stream between the stopnets at the polluted station. It was assumed that the efficiency of capture of introduced (marked) fish was the same as that for resident (unmarked) fish and the population of resident fish was estimated from the equation

$$N = \frac{UM}{R} \qquad (1)$$

where N is the resident fish population, U the resident fish caught, M the marked introduced fish at large, and R the marked introduced fish recaptured. The marked fish were returned to the sample area after each run, and several replicate fishings were carried out. Since the resident fish were not returned, the population estimates calculated from Eqn. 1 after every fishing run except the first, had, of course, to be increased by the total number of unmarked fish removed from the fishing area during all the previous fishing runs. The final estimate of population was obtained by averaging all the population estimates made from the separate trials. The assumption of equal catchability in the method seems to be justified because there is good agreement between estimates based on this method and those made at the same time at some stations by the catch-effort method, which is not influenced by any difference in catchability between marked introduced fish and unmarked residents (TABLE 4). Therefore estimates of the population densities in control streams made by the catch-effort method may fairly be compared with estimates in polluted streams made by the mark-recapture method.

VOL. 5 1961

TABLE 4. COMPARISON OF CATCH-EFFORT AND MARKED-INTRODUCED FISH METHODS OF ESTIMATING THE NUMBER OF RESIDENT TROUT BETWEEN STOPNETS

River system	Station No.	Population estimates	
		Catch-effort method	Method based on recapture of introduced fish
Fal	6	6	10
,,	7	12	11
,,	11	8	13
Par	14	7	11
,,	21	4	6
Camel	24	20	23
,,	25	38	41

Although the three methods for estimating populations gave very similar results, it was not known how these compared with the numbers actually present in a stretch. To examine this, 24 marked fish were put between stopnets at Station 14. By re-marking and returning those marked fish which were subsequently caught, the introduced

TABLE 5. ESTIMATES OF TROUT POPULATIONS IN SOME CORNISH STREAMS. (Catch-effort method used at control stations except Stations 1 and 19; mark-recapture method used at all polluted stations and Stations 1 and 19)

River system	Station No.	Trout/1000 ft²	No. of replicate runs	No. of runs when no resident fish were caught during mark-recapture estimates
	Control stations			
Tresillian	1	18	6	0
,,	2	23	5	–
,,	3	25	3	–
Par	12	27	2	–
,,	17	23	5	–
,,	18	19	8	–
,,	19	16	4	0
Fal	4	15	7	–
,,	5	21	3	–
Camel	26	19	3	–
Average		20·6		
	Polluted stations			
Par	13	3	3	1
,,	14	5	4	1
,,	15	2	2	1
,,	16	3	2	0
,,	20	2	2	0
,,	21	3	4	2
,,	23	4	2	1
Average		3		
Fal	6	3	2	1
,,	7	6	4	0
,,	10	0·1	4	4
,,	11	3	5	2
Average		3·0–3·3		
Camel	24	34	3	0
,,	25	21	3	0
Average		27·5		

population was estimated by CHAPMAN's method and, by neglecting recaptures of re-marked fish, the population was also estimated by the catch-effort method. The estimates by the two methods were 23 and 20, respectively.

Electric stunning is rather more efficient with large fish than with small and this

could have introduced some error into the population estimates. The methods used, however, seem to have given reasonably reliable results, and although confidence limits have not been calculated for the data from all stations, it appears that the true values for the populations will generally have been within 70 to 150 per cent of the estimated values.

Numbers of adult trout

The population densities of brown trout at most sampling stations are given in TABLE 5. In the lightly polluted parts of the Camel system, the densities were similar to those at control stations, but in the heavily polluted Par and Fal the densities were, on average, only one-seventh of the controls. These differences are too large and too consistent to be due to errors in estimating the populations.

Electric fishing was also carried out at Stations 8, 9, and 22. In each case no resident fish were caught, but the results have not been entered in TABLE 5 because the absence of fish may not have been due solely to the presence of china-clay wastes. Stations 8 and 9 may sometimes have been subjected to pollution by untreated sewage and other discharges, while at Station 22 the river fell so swiftly over a steep and rocky incline that it seemed unlikely that it would have supported a resident fish population even if the water had been clean.

Occurrence of trout fry

At most of the biological sampling stations, random collections were made with a hand net, usually for 5 minutes at three places within the sample area. Trout fry—about 4 months old—were found at five of the seven control stations examined in this way, but none was caught at any of the twelve polluted stations. No fry were observed in polluted stretches during electrical fishing either, so it is very probable that trout did not breed in the polluted areas of these rivers.

VOL.
5
1961

Age composition of the trout population

The ages of 100 trout—about half the number caught during the survey—were estimated by reading their scales. The majority were found to be less than 3 years old, and the numbers of one- and two-year-old fish from the survey areas are given in TABLE 6.

It was found that 80 per cent of the trout whose scales indicated that they were one year old were less than 12 cm long, while the same percentage of two-year-olds were under 18 cm, so it was possible to estimate age from body length. All the fish caught during the survey had been measured, and the age distributions as estimated from lengths are also given in TABLE 6. Although these distributions are based on a larger sample, there is less certainty that the age of each individual was correctly estimated. Apart from the absence of fish younger than one year from the polluted stretches and the small number of trout from the polluted Par showing two winter bands on their scales, there were apparently no important differences between the age structures of the populations in the clean and polluted parts of the same river, but there were distinct differences in age composition between the different river systems. The River Fal is particularly interesting, for there two-year-old trout apparently outnumbered the one-year-old fish.

Size of trout

From measurements of the growth rings on the scales, and of body length at the time of capture, estimates were made of the lengths of one- and two-year-old trout at the end of previous winters (TABLE 7). Lengths were similar in both clean and polluted reaches and are close to those for trout in a Devon stream (HORTON, 1958), and in Windermere (ALLEN, 1938).

TABLE 6. NUMBER OF ONE- AND TWO-YEAR-OLD TROUT CAUGHT IN SOME CORNISH STREAMS

River system	Conditions	Number of trout			
		Age estimated from			
		Scale readings		Length measurements	
		1 yr+	2 yr+	1 yr+	2 yr+
Par	Clean	11	10	23	24
	Polluted	12	3	12	11
Fal	Clean	3	9	13	27
	Polluted	6	16	7	15
Camel	Polluted	7	10	34	18

TABLE 7. LENGTHS OF TROUT OF VARIOUS AGES, ESTIMATED FROM SCALE READINGS AND LENGTHS AT TIME OF CAPTURE

River system	Condition	Age at time of capture		
		1 yr+	2 yr+	
		Average length (cm) at end of		
		1st winter	1st winter	2nd winter
Fal and Par	Unpolluted	6·9	7·1	14·2
Fal and Par	Polluted	7·7	7·2	13·0
Camel	Polluted	7·1	6·0	12·7

With fish of one species, weight usually varies in direct proportion to approximately the third power of length, so the relation between log weight and log length is linear with a regression coefficient, or slope, of about 3. These regression lines can be used to compare the condition, or "plumpness", of different populations (FIG. 6). Statistical examination of the lengths and weights of the trout caught in this survey did not reveal any difference in the conditions of the fish in polluted and unpolluted parts of the Par and Fal, which suggests that trout in the polluted stretches grew normally and that there must have been sufficient food available to maintain the small populations

there in good condition. Older fish from the polluted Camel, however, were slightly, but significantly, thinner than those from the Par and Fal, but unfortunately no data from the clean parts of the Camel are available for comparison.

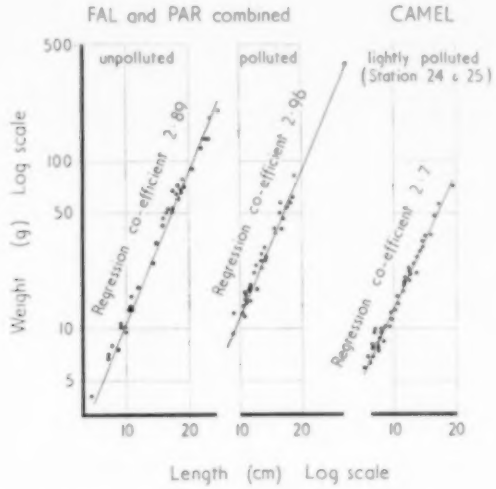


FIG. 6. Relation between logarithm of weight and logarithm of length of brown trout in some Cornish streams.

VOL. 5 1961

EXAMINATION OF TROUT GILLS

HERBERT and MERKENS (1961) have reported that the gills of some rainbow trout kept in aquaria containing mineral solids in suspension were abnormal in histological structure: in places the epithelial cells of the secondary lamellae were thickened, and fusion between adjacent lamellae sometimes occurred. Two gill arches were removed from many of the fish caught during this survey, fixed in Bouin's solution, and later examined to see whether similar histological changes had occurred. The results are summarized in TABLE 8, and two photomicrographs are shown in FIG. 7. One is of a slightly affected gill showing some fusion but little or no thickening, and the other is a severely affected patch of a moderately affected gill in which epithelial thickening is the dominant feature.

In some cases where most of the gill was normal it was difficult to decide whether small patches of apparently fused or thickened epithelium were genuine or due to some histological artefact. These cases were classified as normal, and only cases where there was little doubt that the effect was genuine have been classed as affected in TABLE 8. Several doubtful gills from the polluted parts of the River Par were classed as normal and it is possible that slightly affected gills were more common there than the Table suggests. However, there is no doubt that by whatever standard the sections are judged, affected gills were more common in the sample from the polluted Fal than from the polluted Par. This is surprising because the concentration of solid matter was higher in the Par. However, the proportion of fish which were two years old was higher in the

TABLE 8. OCCURRENCE OF TROUT GILLS SHOWING THICKENING OR FUSION OF SECONDARY LAMELLAE IN SOME CORNISH STREAMS

River system	Station No.	No. of gill arches examined	No. of gill arches affected
	Control Stations		
Par	12	4	0
Fal	5	6	0
	Polluted Stations		
Par	13	3	0
,,	15	3	0
,,	16	6	0
,,	20	2	0
,,	23	3	0
Fal	6	8	3
,,	7	11	4
,,	11	15	4
Camel	24	2	0

Fal than in the Par, so more trout from the Fal may have been in contact with the suspensions for longer than those from the Par.

All trout caught in clean streams were of normal appearance but all those from polluted waters including the Camel were noticeably light in colour, and much paler than normal light-adapted fish. A similar condition has been observed in fish kept for long periods in suspensions of china clay under laboratory conditions (HERBERT and MERKENS 1961).

AVAILABILITY OF FISH FOOD

Methods

At the majority of sites where the population of trout was estimated, measurements were made of the abundance of potential trout food. The samples were taken usually about 24 hours after the area had been fished. In the streams investigated, which contained very little rooted vegetation, the most important possible sources of food for the trout were the fauna on the bed of the stream, and that which was carried on the surface and in the body of the water as drift.

At each site, invertebrates from pebbly, stony, or rocky portions of the bed of the stream were collected with a stream-bottom sampler (SURBER 1937) fitted with a conical collecting bag of nylon bolting cloth of 20 mesh to the inch. Portions of the bed which consisted of fine gravel, sand, silt, or mud were sampled with a scoop over an area of 2·18 ft² and to a depth of about 1 in. Normally, 3 samples of each sort were taken just above, just below, and in the middle of the area which had been fished. Then the whole area was surveyed to determine the relative areas of "Surber bottom" and "scoop bottom". The samples were preserved in formalin and the wet weights of the invertebrates in each collection were determined later. Estimates of the wet weight of the standing crop of invertebrate animals per unit area of stream bed were then calculated.

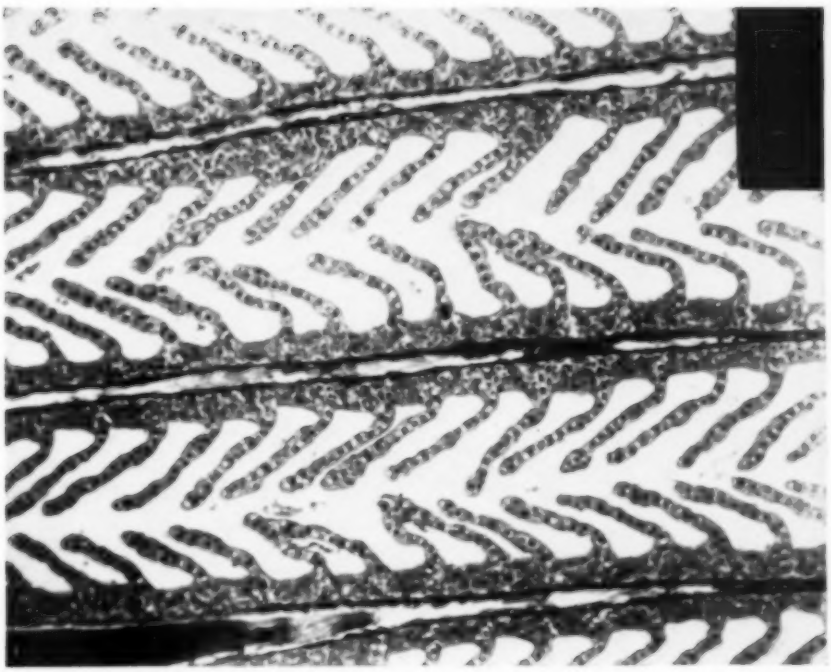


Fig. 7. Two brown-trout gills from reaches polluted with china-clay wastes.
(a) Slightly affected gill from Station 7 showing some fusion of lamellae.

VOL.
5
1961

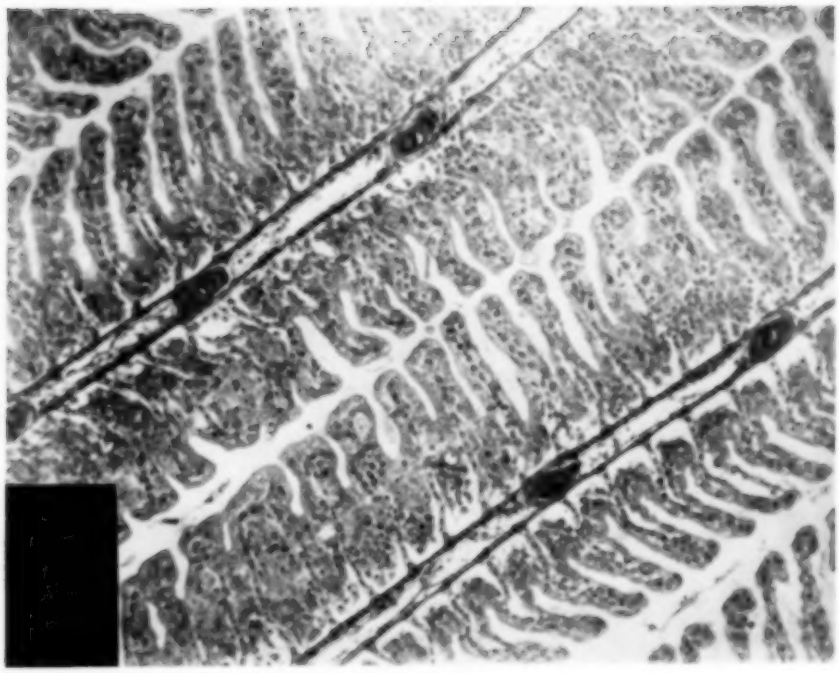


FIG. 7.

(b) Severely affected patch of gill from Station 11, showing thickening of respiratory epithelium.

The animal food coming down as drift was estimated by using a drift net (also made of nylon bolting cloth of 20 mesh per inch) for a period of 1 hr at each station, and the volume of water passing through the mouth of the net was measured with a Watts current meter. The collections were examined in the same way as the samples of bottom

TABLE 9. ABUNDANCE OF INVERTEBRATE BOTTOM FAUNA IN SOME CORNISH STREAMS

River system	Station No.	Abundance of invertebrate fauna on stream bed (g/1000 ft²)
	Control stations	
Tresillian	1	1081
,,	2	354
Par	12	333
,,	17	1062
,,	19	669
Fal	4	400
,,	5	807
Camel	26	411
Average		640
	Polluted stations	
Par	13	11
,,	15	46
,,	16	59
,,	20	27
,,	21	21
,,	23	54
Average		36
Fal	6	250
,,	7	129
,,	8	143
,,	9	192
,,	11	197
Average		182
Camel	25	572

fauna. Depths were measured to the nearest ½ in. and current velocities determined at 1–ft intervals across a convenient cross-section at each Station, and the flow of the river was calculated from those measurements.

Bottom fauna

TABLE 9 shows that the abundance of bottom fauna in the control streams was on average 3·3 times greater than in the polluted part of the Fal and 19 times greater than in the polluted part of the Par. The greater abundance of bottom fauna in the polluted

VOL. 5 1961

parts of the Fal was probably a consequence of the much smaller concentration of suspended matter there. In the lightly polluted tributary of the Camel, the abundance of bottom fauna was about the same as in the control streams.

Invertebrate drift

In Table 10 the abundance of invertebrate drift is expressed in three ways.

Table 10. Abundance of invertebrate drift in some Cornish streams

River system	Station No.	Invertebrate drift		
		Per unit volume of water (mg/m³)	Passing through measured cross-section (g/hr)	Passing above unit width of stream bed (g/ft hr)
	Control stations			
Tresillian	1	186	5	0·9
,,	2	37	4	0·4
Par	12	9	2	0·2
,,	17	150	13	1·3
,,	19	39	1	0·3
Fal	4	55	9	0·8
,,	5	15	3	0·2
Camel	26	126	14	1·6
Average		77	6	0·7
	Polluted stations			
Par	13	13	1	0·2
,,	15	52	8	0·8
,,	16	96	21	4·3
,,	20	88	13	0·9
,,	21	80	13	0·8
,,	23	72	18	1·5
Average		67	13	1·4
Fal	6	55	15	1·1
,,	7	8	2	0·2
,,	8	81	6	1·2
,,	9	15	5	0·6
,,	11	81	30	1·8
Average		48	12	1·0
Camel	25	14	1	0·1

Although there was considerable variation in the abundance from station to station, there was no marked and regular difference between the control streams and those polluted with clay wastes. Approximate estimates of the relative proportions (as

judged by eye) of terrestrial and aquatic organisms have been made and are given in TABLE 11. Both truly terrestrial animals and the terrestrial stages of aquatic forms were classed as terrestrial.

These assessments were made by eye because it was desired to compare them with similar estimates of relative abundance in the stomach contents of fish taken from these rivers. Often, part of this material is broken and partly digested and only some of

TABLE 11. APPROXIMATE PROPORTIONS (by volume) OF TERRESTRIAL INVERTEBRATES IN TROUT STOMACHS, IN DRIFT, AND ON THE STREAM BEDS. MAY 1960

River system	Station No.	Proportion terrestrial/total		
		In stomachs	In drift	On beds
	Control stations			
Tresillian	1	–	0·3	0·0
,,	2	–	0·3	0·02
Fal	4	–	0·4	0·0
,,	5	0·8	0·4	0·0
Par	12	0·5	0·1	0·05
,,	17	0·05	0·05	0·0
,,	19	–	0·6	0·0
Camel	26	–	0·7	0·1
	Polluted stations			
Fal	6	0·9	0·67	0·2
,,	7	0·7	0·50	0·45
,,	11	0·3	0·50	0·05
Par	13	0·7	0·4	0·10
,,	15	0·8	0·5	0·20
,,	16	0·4	0·9	0·35
,,	20	0·5	0·4	0·31
,,	21	1·0	0·9	0·45
,,	23	0·7	0·95	0·3
Camel	24	–	–	0·1
,,	25	0·05	0·7	0·15

it is identifiable. Since it is possible in the imagination to reconstruct the whole creature from the pieces, visual estimates of volume are probably better than measuring only the identifiable fragments.

Most samples of invertebrate drift contained a substantial proportion of terrestrial creatures which had obviously fallen into the water. In the control streams it appeared that the majority of the drift consisted of aquatic animals, while in the polluted streams terrestrial forms were generally in the majority; probably this was a consequence of the less abundant bottom fauna in the polluted stretches.

VOL. 5 1961

Detailed classification and enumeration of the aquatic invertebrates in the collections were not attempted, and only a brief description of the fauna will be given. The most abundant aquatic insects were the nymphs and pupae of mayflies, stoneflies, caddisflies and chironomids, with some dragonfly and damselfly larvae. These groups were represented in both clean and polluted reaches. *Gammarus* was relatively abundant in both clean and polluted reaches of the River Par, but was not observed in any of the collections from the River Fal. Oligochaete worms were found in clean and polluted parts of all rivers, but gastropods were found only in the unpolluted waters; perhaps the reason is that encrusting algae on which they could graze were virtually absent from the polluted streams. Bivalves were found at some control stations but not at any polluted stations. Apart from the absence of molluscs in polluted stretches, the main effect of the pollution on the fauna was apparently to reduce its abundance, but not greatly to alter its gross composition.

Examination of stomach contents

The stomach contents of trout caught from most of the stations on the polluted rivers, and from a few caught in control streams were sorted into groups according to whether they were aquatic or terrestrial in origin. Approximate visual estimates were then made of the proportion of terrestrial organisms in the food of the fish from those stations where the stomach contents were both sufficiently abundant and in a sufficiently intact condition for reasonably reliable estimates to be made. The relative proportion of terrestrial invertebrates varied considerably from station to station (Table 11) but it seems that in the control streams the diet consisted on average of equal proportions of aquatic and terrestrial invertebrates, while in the polluted streams terrestrial forms made up the larger proportion of the food eaten by the trout. Quantitative assessments of the amount of food eaten were not attempted, but there was considerable variation in the degree of fullness of stomachs both from clean and polluted stations. However the impression was gained that trout at the polluted stations managed to get as much to eat as similar individuals at most control stations.

DISCUSSION

Suspended solids may have brought about the observed reduction in the numbers of trout in the polluted Par and Fal by making the environment adverse in one or all of a variety of ways: by affecting breeding, by reducing the food supply, and by exerting some harmful effect directly upon the fish.

Brown trout cover their eggs with gravel after they are fertilized, and for successful development a current of well oxygenated water must pass through the gravel (Stuart, 1953). Only a brief inspection is needed to show that the gravelly parts of the beds of the polluted Par and Fal are so clogged with china-clay wastes that suitable spawning grounds are virtually non-existent, and since no trout fry were ever observed in these polluted stretches it is practically certain that trout cannot breed in them. The trout found there must have migrated from the clean headwaters—in some cases through several miles of polluted river—and this may have tended to keep the population densities low. Perhaps trout can occur in the polluted Par and Fal only because these rivers have extensive unpolluted headwaters and tributaries.

The evidence from a number of sources (Pentelow, 1932; Neill, 1938; Horton, 1958;

MÜLLER, 1954; NILSSON, 1957) suggests that invertebrate drift is an important source of food for the brown trout in streams, at least during the summer months. It is possible that the trout in Cornwall fed rather more on drift than from the stream beds, but they did not feed enclusively on drift—some had eaten caddis larvae with heavy stony cases which did not occur in the drift collections and must have been taken from the stream bed—so the scarcity of bottom fauna in the polluted stretches must have meant that there was less food for trout there.

By whatever means the trout sought out their food, roughly half of it consisted of terrestrial forms which must have fallen into the rivers. Consideration of the proportions of aquatic to terrestrial forms in the drift and on the bottom and of the total abundance of invertebrates from these sources shows that terrestrial invertebrates were about equally plentiful in both clean and polluted reaches. Thus a ratio so large as 7 to 1 in trout population densities would not have been expected if abundance of food had been the only important factor governing population size. However, the food supply during May may not be the deciding factor. PYEFINCH (1960) reports that the stomachs of Loch Tummel trout contained a much smaller proportion of terrestrial insects in winter than in summer, and "permanent" members of the bottom fauna may be a more important source of trout food in winter than are terrestrial organisms. The trout populations in the polluted Cornish rivers may be small because there is not enough food to support a larger number through the winter months.

From PENTELOW's (1949) work on china clay in Cornwall, and recent reviews (DOUDOROFF, 1957 and HYNES, 1960), the most widely held view appears to be that suspended solids in rivers harm fish indirectly by damaging food supplies or spawning grounds, and that they do not harm fish directly unless present at very high concentrations (several g/100 ml). But it has recently been shown that the chances of survival of rainbow trout in laboratory aquaria are adversely affected by continuous contact with 810 and 270 p.p.m. mineral suspended solids, and perhaps to a lesser extent by 90 p.p.m. (although some individuals can survive these concentrations for several months at least), whereas 30 p.p.m. is apparently harmless (HERBERT and MERKENS, 1961). These findings are not at variance with the results obtained from the Cornish rivers. Moreover, the gills of some trout caught in the polluted parts of the Fal showed changes similar to those observed in the laboratory studies. It is therefore possible that the chances of survival of trout in the polluted Par and Fal were reduced to some extent by the direct action of the solids in suspension.

Thus, silting of spawning sites, reduction in food supply and a directly adverse action of the solids on the fish, seem all to have played a part in reducing the trout populations of the Cornish streams polluted with china-clay wastes, but from this short survey the relative importance of each of these cannot be assessed. However, the conclusion of greatest practical importance is that the suspended solids from china-clay workings in concentrations of 1000 p.p.m. markedly reduced the abundance of brown trout, although they did not eliminate this species completely, whereas suspensions of about 60 p.p.m. had no adverse effect on the trout population.

Acknowledgements—This paper is published by permission of the Ministry of Agriculture, Fisheries and Food and the Department of Scientific and Industrial Research

We wish to acknowledge the great help given in many ways by Mr. R. E. Taylor, Fisheries Officer of the Cornwall River Board, and his staff throughout the survey and by the many persons who

VOL. 5 1961

allowed us access to the streams. Mr. A. B. WHEATLAND of the Water Pollution Research Laboratory helped to plan the chemical sampling programme and to supervise the analysis of samples. Mr. W. G. HARTLEY of the Ministry of Agriculture, Fisheries and Food kindly assisted in reading trout scales, Mr. P. A. HURSEY, also of the Ministry, assisted in the field work, and Messrs. B. J. BORNE, H. T. MANN, A. M. BRUCE, A. COCKBURN, W. F. ROSS, and A. C. WAKEFORD of the Water Pollution Research Laboratory assisted in the examination of samples.

REFERENCES

ALLEN K. R. (1938) Some observations on the biology of the trout (*Salmo trutta*) in Windermere. *J. Anim. Ecol.* **7,** 333–349.

BETZ J. D. and NOLL C. A. (1950) Total-hardness determination by direct colorimetric titration. *J. Amer. Wat. Wks Assoc.* **42,** 49–56.

BRITISH STANDARD 1377:1948. Methods of test for soil classification and composition, p. 32. British Standards Institution, London.

CHAPMAN D. G. (1952) Inverse, multiple and sequential sample censuses. *Biometrics* **8,** 286–306.

DELURY D. B. (1958) The estimation of population size by a marking and recapture procedure. *Biometrics* **15,** 19–25.

DIEHL H., GOETZ C. A. and HACH C. C. (1950) The versenate titration for total hardness. *J. Amer. Wat. Wks Assoc.* **42,** 40–48.

DOUDOROFF P. (1957) *Water quality requirements of fishes and effects of toxic substances. The Physiology of Fishes.* (Ed. M. E. BROWN). Academic Press, New York, pp. 403–430.

EDEN G. E. and MELBOURNE K. V. (1959) A battery-operated water sampler. *Chem. and Industr.* 220.

ELSON T. F. (1960) Personal Communication.

HERBERT D. W. M. and MERKENS J. C. (1961) The effect of suspended mineral solids on the survival of trout. *Int. J. Air Wat. Poll.* **5,** 46–55.

HORTON P. A. (1958) The bionomics of brown trout in a Dartmoor stream. Ph.D. Thesis. University of Exeter.

HYNES H. B. N. (1960). *The biology of polluted waters.* The University Press, Liverpool.

MINISTRY OF HOUSING AND LOCAL GOVERNMENT (1956) *Methods of chemical analysis as applied to sewage and sewage effluents.* H.M. Stationery Office, London.

MÜLLER K. (1954) Investigations on the organic drift in North Swedish streams. *Inst. Freshwat. Res. Drottningholm Rept.* No. 35. 133–148.

NEILL R. M. (1938) The food and feeding of the brown trout (*Salmo trutta* L.) in relation to the organic environment. *Trans. Roy. Soc. Edinburgh* **59,** (18), 481–520.

NILSSON N. A. (1957) On the feeding habits of trout in a stream of Northern Sweden. *Inst. Freshwat. Res. Drottningholm, Rept.* No. 38.

PENTELOW F. T. K. (1932) The food of the brown trout (*Salmo trutta* L.). *J. Anim. Ecol.* **1,** (2), 101–107.

PENTELOW F. T. K. (1949) Fisheries and pollution from china clay works. *Rept. Salm. Freshwat. Fish. Lond.* **31,** 4 pp.

PYEFINCH K. A. (1960) *Trout in Scotland.* H.M. Stationery Office, Edinburgh.

STUART T. A. (1953) Spawning, migration, reproduction and young stages of loch trout (*Salmo trutta* L.). *Sci. Invest. Freshwat. Fish. Scot.* **5,** 39 pp.

SURBER E. W. (1937) Rainbow trout and bottom fauna production in one mile of stream. *Trans. Amer. Fish. Soc.* **66,** 193–202.

VOGEL A. I. (1957) *A Textbook of Quantitative Inorganic Analysis.* 2nd Ed. Longmans, Green, London.

Int. J. Air Wat. Poll. Pergamon Press 1961. Vol 5, No. 1, p. 75. Printed in Great Britain.

LETTER TO THE EDITOR

(*Received* 20 *February*, 1961)

Aerodynamic Capture of Particles

The latter part of my comment, Vol. 3, nos. 1–3, p. 59, was omitted from the printed text and its point lost. This was to demonstrate that viscous forces will prevent an impinging sphere from making contact with a plane surface or with another sphere, and hence that hydrodynamical theory alone cannot provide a complete explanation of collision and capture processes.

An expression was derived for the force F required to extrude a film of viscous fluid from the gap between a sphere and a plane as the sphere approaches with velocity v, namely

$$F = 6\pi a^2 \eta v/y$$

where a is the radius of the sphere, η the viscosity of the fluid and y the instantaneous distance between sphere and plane ($y \ll a$). This force, acting on the sphere of mass m, decelerates its motion and so we have:—

$$F = mv\frac{dv}{dy}.$$

The original argument continued by substituting for F and integrating, giving

$$v = v_3 + \int_{y_0}^{y} \frac{dv}{dy}\,dy = v_0 - \frac{6\pi a^2 \eta}{m} \ln yo/y$$

where v_0 is the velocity at distance y_0.

The sphere is therefore brought to rest at a distance y^1 from the plane such that

$$\ln yo/y^1 = \frac{mv_0}{6\pi a^2 \eta}, \text{ or } \frac{2\rho a v_0}{9\eta},$$

ρ being the density of the sphere.

Thus the sphere will never make contact with the plane. A similar result would be obtained for the approach of one sphere to another.

If practical values are substituted in the above equation y^1 comes out very small for airborne particles and it is clear that other factors such as surface roughness and molecular attraction will intervene. The result suggests that such factors, which are not usually introduced into the theoretical models, may in practice be significant.

Similar forces to those discussed above will operate when particles separate.

W. H. WALTON

VOL. 5 1961

Int. J. Air Wat. Poll. Pergamon Press 1961. Vol. 5, No. 1, pp. 76–79. Printed in Great Britain.

ABSTRACTS OF PAPERS PUBLISHED IN *STAUB*

FOURTH QUARTER 1960

Improvements in the routine measurement of particle size distributions for fine dusts sampled with a thermal precipitator and using an electron microscope. I. WESTERBOER (Münster; Abteilung fur Medizinische Electronenmikroskopie der Universität) *Staub*, vol. **20**, pp. 361–364, 11 refs.

When dust is sampled with the Casella thermal precipitator a gradation of particle sizes is found at right angles to the hot wire, and a series of electron micrographs have to be analyzed to obtain an average particle size distribution. To minimize this effect, so that a single electron micrograph will completely cover the particle size distribution, a modified thermal precipitator (with a finer wire—60 μ dia. compared with 250 μ dia. and a smaller slit—0·31 × 4·5 mm cf 0·55 × 10 mm) has been developed. The earlier Walkenhorst modification (*Staub*, **40**, 241, 1955) is also suitable. The method of membrane preparation (a modification of the CARTWRIGHT and SKIDMORE (S.M.R.E. Report 79 (1953) method) is described in detail.

The beta method; a new stack sampling procedure. K. BREITLING (Dresden). *Staub*, vol. **20**, pp. 364–335, 3 refs.

To overcome the sampling difficulties due to stack load variations, a system of multiple sampling and velocity measurements is discussed which will, under certain circumstances, be effective.

Dust measurement with membrane filters in Czechoslovakia. J. SIMECEK and DR. L. OPPL (Prague; Institute for Industrial Hygiene and Medicine) *Staub*, vol. **20**, pp. 366–368, 13 refs.

Membrane filters with a gravimetric efficiency of almost 100 per cent are used for dust concentration measurements in Czechoslovak Industry. Particle concentrations are also found from the mean particle size distributions for various dust types.

New M.A.C. values for the U.S.S.R. DR. H. KETTNER (Berlin). *Staub*, vol. **20**, pp. 369–370, 2 refs.

Discussion Meeting on Dust Filters (29–30th November 1960, Bad Godesberg).

A method of measuring radioactive aerosol concentrations with filters. DR. K. H. WEBER and W. WISCH (Dresden; VEB Vakutronik). *Staub*, vol. **20**, pp. 393–398, 11 refs.

Radioactive aerosol concentrations can be measured by the enrichment of activity in a fibre filter through which the gases containing the aerosol are being drawn. The degree of activity can be measured with a comparative technique using filter materials with known absorbed concentrations and similar geometry. Details are discussed of methods for the evaluation of radioactive aerosol concentrations, and the corrections which have to be made to allow for the exponential falling off in activity of the aerosol and to account for filter inefficiencies.

Classification of dust measuring techniques using membrane filters. DR. K. SPURNY (Prague: Institute for Physical Chemistry, Czechoslovak Academy of Science). *Staub*, vol. **20**, pp. 398–400, 34 refs.

The production of membrane filters with radioactive markers and their application in dust technology. DR. K. SPURNY and DR. G. KUBIE (Prague; Institute for Physical Chemistry and Mining, Czechoslovak Academy of Science). *Staub*, vol. **20**, pp. 400–402, 7 refs.

A method for the production of a membrane filter containing radioactive nickel. These filters have an activity of about 0·05 mc/cm². When they are used for aerosol filtration, the collected dust acts as an absorbent for β radiation, and the reduction of filter activity can be used as a measure of dust absorbed The change in radioactivity can be used as the basis for an automatic dust concentration measuring instrument.

The use of absorption of β recoil in the measurement of deposited aerosol samples. Dr. C. Jech (Prague; Institute for Physical Chemistry, Czechoslovak Academy of Science). *Staub*, vol. **20,** p. 403–404, 2 refs.

It was found experimentally that the recoil absorption depended not only on the weight of particles absorbed, but also on particle size. The same weight of particles will act as a more efficient absorber when finely dispersed on the filter surface than when concentrated in larger grains. This can be expressed mathematically and the relation applied to evaluating the mean particle size.

The routine determination of free silica. Prof. K. G. Schmidt (Bonn, Staubforschungsinstitut des Hauptverbandes der gewerblichen Berufsgenossenschaften e.V.). *Staub*, vol. **20,** pp. 404–411, 11 refs.

The method for free silica determination developed by the *Staubforschungsinstitut* in Bonn is described in detail. This method, which uses the decomposition of phosphoric acid as its major step, is the most accurate available for determining free silica in rocks and mineral products.

The coagulation and dispersion of fine grained dry minerals. W. Süsse (Berlin; Fakultät für Bergbau und Hüttenwesen der Technischen Universität; Dissertation). *Staub*, vol. **20,** pp. 429–438, 11 refs.

Experiments were carried out to determine the effect of surface active agents on the dispersion and coagulation of aerosols. This can be achieved by varying the type and quantity of the additive. It is thought that the addition of the surface active material effects the electric charge on the particles.

The reaction of SO_3 with cellulose filters (provisional report). F. Baum (Stuttgart, Institut für Technische Physik). *Staub*, vol. **20,** pp. 438–439, 2 refs.

Gases containing sulphur trioxide react with the cellulose molecule chains, dehydrating the molecules.

The vaporization of liquid fogs interfering in dust measurements with a tyndalloscope. Dr. J. Olaf (Essen-Kray; Bergbau-Forschung G.m.b.H., Research Institute of the Coal Mining Association). *Staub*, vol. **20,** pp. 440–445, 15 refs.

Underground dust measurements with a konimeter or tyndalloscope are made more difficult because of the interference caused by fogs from saline solutions in the mine. These mists can be dispersed by either reducing the humidity or increasing the temperature in the chamber. Suitable apparatus using both heating and drying techniques developed by the authors are described.

VOL. 5 1961

FIRST QUARTER 1961

The Development of routine X-Ray methods of quartz determination in coalmine dusts. Dr. R.-W. Schliephake. (Essen: Petrographisch-Mineralogisches Laboratorium der Bergbau Forschung G.m.b.H., Forschungsinstitut des Steinkohlenbergbauvereins). *Staub*, vol. **21,** pp. 1–5, 14 refs.

A routine X-ray technique for quartz determination in mine-dusts has been evolved. Statistical analysis of the results indicates that the method is comparable with chemical analysis.

The determination of emission limits by the V.D.I. clean air commission. Dr. K. Schwarz. (Essen: T.U.V.). *Staub*, vol. **21,** pp. 6–7, 10 refs.

Data requirements for the selection of dedusting equipment (particularly inertial separators and possible plant efficiency limitations.

R. Nagel and Dr. R. Ibing. *Staub*, vol. **21,** pp. 8–15, 30 Refs.

It is suggested that gas-cleaning plant customers state their requirements in terms of grade (or fractional) efficiency curves, which can then be compared with the efficiency curves obtained with similar conditions on available plant.

Two methods of sampling atmospheric dust coming from a definite direction. R. Kröner. (Neuölsburg). *Staub*, vol. **21,** pp. 15–16.

U.S. Pat. 2386282 (9th October, 1945) and German Patent Application 1091366 (20th October, 1960).

Report of a meeting on aerosol techniques held at Mainz, 27th and 28th October, 1960. (REV. by K. H. SCHMITT). *Staub*, vol. **21,** pp. 16–19.

Technical meeting on clean air

(Wiesbaden, 21st-23rd September, 1960). 29 papers were presented at this conference. As these papers were largely reviews, only their titles and authors will be given here.

1st Session: The action of air pollutants:

1. A survey of the activities of the V.D.I. Clean Air Commission. (Introductory Lecture). DR. K. SCHWARZ. (Essen: Technischer Überwachungsverein). *Staub*, vol. **21,** pp. 39–43, 7 refs.

2. Clean Air, its supervision and the law. H. STEPHANY. (Bonn: Federal Ministry for Labour and Social Order). *Staub*, vol. **21,** pp. 43-47.

3. Health and city atmospheres. Prof. H. O. HETTCHE. (Hamburg: Institute for Hygiene). *Staub*, vol. **21,** pp. 48-50.

4. The analysis of the action of sulphur dioxide on plants in field and laboratory experimentation. Prof. K. EGLE. (Frankfurt a.M.: Botanical Institute of the University). *Staub*, vol. **21,** pp. 51–52.

5. The analysis of the action of sulphur dioxide on plants in the laboratory. DR. H. VAN HAUT. (Essen-Bredeney: Forschungsinstitut für Luftreinhaltung e.V.) *Staub*, vol. **21,** pp. 52–56, 8 refs.

6. The action of sulphur dioxide on plants. DR. R. ZAHN. (Frankfurt (Main)—Höchst: Farbwerke Hoechst A.G.) *Staub*, vol. **21,** pp. 56–60, 7 refs.

7. Results of the field experiment Biersdorf/Sieg to determine the action of sulphur dioxide on vegetation. DR. R. GUDERIAN. (Essen: Forschungsinstitut für Luftreinhaltung e.V.). *Staub*, vol. **21,** pp. 60-61.

8. The use of air analyses in determining the action of sulphur dioxide on vegetation. DR. H. STRATMANN. (Essen: Forschungsinstitut für Luftreinhaltung e.V.). *Staub*, vol. **21,** pp. 61–64, 8 refs.

9. The action of hydrogen sulphide and carbon disulphide. DR. H. PETRI. (Berlin-Dahlem: Bundesgesundheitsamt, Institut für Wasser, Boden und Lufthygiene). *Staub*, vol. **21,** pp. 64–68.

2nd Session: The emission of smoke and gases

10. Dust Emission in Firing Boilers. A. BACHMAIR. (Essen: Vereinigung der Grosskesselbesitzer). *Staub*, vol. **21,** pp. 68–70, 1 ref.

11. Sulphur dioxide emission. Dr. K. SCHWARZ. (Essen: Technischer Überwachungsverein). *Staub*, vol. **21,** pp. 71–77, 29 refs.

12. The emission of smoke and gases in iron and steel Works and Foundries. DR. K. GUTHMANN. (Düsseldorf: Verein Deutscher Eisenhüttenleute). *Staub*, vol. 21, pp. 78–82, 12 refs.

13. The Emission of Smoke and Gases in the Non-Ferrous Metallurgical Industry. G. OSWALD. (Braubach/Rhein). *Staub*, vol. **21,** pp. 82–84, 2 refs.

14. Emissions from the Chemical Industry. DR. R. PISTOR. (Leverkusen: Farbenfabricken Bayer A.G.) *Staub*, vol. **21,** pp. 84–97, 9 refs.

15. The origin and causes of air pollution in the Petroleum Industry. DR. W. BECKER. (Gelsenkirchen, Gelsenberg Benzin A.G.). *Staub*, vol. **21,** pp. 88–89.

16. The emission of smoke and gases in Coke Ovens and Gas Works. DR. E. FRICHMELT. (Gelsenkirchen-Buer, Essener Steinkohlenbergwerke A.G.). *Staub*, vol. **21,** pp. 89–91.

17. The emission of smoke and gases in the Cement Industry. E. RUHLAND. (Neubeckum/Westf.: Dyckerhoff Zementwerke A.G. and Elsa Zement und Kalkwerke A.G.). *Staub*, vol. **21,** pp. 91–94.

18. The Emission of Smoke and Gases in Domestic Units. K. LENHART. (Essen: Ruhrkolenberatung G.m.b.H.). *Staub*, vol. **21,** pp. 94–97.

19. Waste gases from diesel engines. DR. M. EITEL. (Stuttgart-Untertürkheim, Daimler-Benz A.G.). *Staub*, vol. **21,** pp. 97–100, 8 refs.

20. Waste gases from spark ignition engines. DR. E. ALT. (Ingoldstadt; Auto-Union G.m.b.H.). *Staub*, vol. **21,** pp. 101–103, 11 refs.

3rd Session: The spreading of smoke and gases

21. The theoretical approach to diffusion. DR. F. WIPPERMANN. (Darmstadt: Meteorological Institute of the Technische Hochschule). *Staub*, vol. **21,** pp. 104–109, 4 refs.

22. Practical application of the diffusion calculation. Dr. W. Klug. (Darmstadt: Meteorological Institute of the Technische Hochschule). *Staub*, vol. **21**, pp. 109–112, 10 refs.

23. The measurement of metorological factors which influence the spread of air pollutants. H.-P. Barthelt. (München, Flughafen Riem: Institut für Flugmeteorologie der Flugwissenschafflichen Forschungsanstalt e.V.). *Staub*, vol. **21**, pp. 112–116, 4 refs.

24. Experiments in the spreading of gases. H. Markusch. (Darmstadt: Meteorological Institute of the Technische Hochschule). *Staub*, vol. **21**, pp. 116–118, 3 refs.

4th Session: The technique of dust and smoke measurement

25. Criteria in the measurement of gaseous pollutants. Dr. H. Stratmann. (Essen: Forschungsinstitut für Luftreinhaltung e.V.). *Staub*, vol. **21**, pp. 118–121, 15 refs.

26. The measurement of gas concentrations in industrial emissions. Dr. H. W. Thoenes. (Essen: Technischer Überwachungsverein Essen e.V.). *Staub*, vol. **21**, pp. 121–124, 7 refs.

27. The measurement of motor car exhaust gases. Prof. H. Luther. (Clausthal: Institut für Brennstoffchemie und Brennstofftechnik der Bergakademie). *Staub*, vol. **21**, pp. 125–131, 27 refs.

28. Dust Concentration Measurement in Air. Dr. D. Hasenclever. (Bonn: Staubforschungsinstitut). *Staub*, vol. **21**, pp. 131–136, 5 refs.

29. Recording dust concentration measurements. Prof. Th. Gast. (Walsum: Laboratorium für Staubphysik und Messtechnik der Bergwerksgesellschaft Walsum). *Staub*, vol. **21**, pp. 136–139, 10 refs.

The concluding address was given by Dr. K. Schwarz.

Optical and electron microscopic investigation of the deposition of dust in the respiratory system. Prof. L. Dautrebande. (Brussels), Dr. H. Beckmann and Dr. W. Walkenhorst. (Bochum: Silikose Forschungsinstitut der Bergbau Berufsgenossenschaft). *Staub*, vol. **61**, 141–151, 32 refs.

The deposition of 6 types of dust (coal dust, indian ink, titanium dioxide, aluminium, ferric oxide and calcium carbonate) in the respiratory system was studied using optical and electronmicroscopic photographs for measurement of particle size and distribution. It was found that the deposition of solid particles depends on the frequency of breathing—the higher the frequency, the lower the effective deposition. For most particles about 90 per cent of those inhaled are deposited in the respiratory tracts and lungs. Particles of indian ink, titanium dioxide and aluminium (max. particle size 0·5 microns) are deposited deep in the lungs. Particles, with wider size spectra, show a lower average particle size in the exhaled air than in the inhaled air. Minimum deposition for coal particles was in the size range 0·4–0·6 microns. For other particles the minimum occurred at 0·25 microns, in approximate agreement with workers on fibrous filters.

The statistical analysis of deposition on adhesive papers. F. Baum and Louise Hermann. (Stuttgart: Institute for Technical Physics). *Staub*, vol. **21**, pp. 151–154, 7 refs.

The qualitative and quantitative evaluation of deposits on sticky papers.

A new technique for measuring the wind-direction dependence of dust deposits. F. Baum. (Stuttgart: Institute for Technical Physics).

An apparatus for exposing sticky paper in one direction (that of the wind) only is described.

Translated and Abstracted by W. Strauss,

Department of Chemical Engineering, University of Melbourne, Australia.

VOL. 5 1961

Int. J. Air Wat. Poll. Pergamon Press 1961. Vol. 5, No. 1, pp. 80–83. Printed in Great Britain.

AIR POLLUTION BIBLIOGRAPHY

WE ARE INDEBTED TO THE COMPILERS OF THE SOURCES FROM WHICH THE TITLES LISTED IN THIS BIBLIOGRAPHY ARE SELECTED. THE NUMBER OF PAPERS PUBLISHED IS SO LARGE THAT WE CONSIDERED A SELECTION OF THE MORE IMPORTANT PAPERS MADE BY ONE WITH GREAT EXPERIENCE IN THE FIELD OF AIR POLLUTION WOULD BE OF CONSIDERABLE VALUE, A TASK KINDLY UNDERTAKEN BY MR. T. F. HURLEY.

ABSTRACTS OF THE PAPERS LISTED CAN BE FOUND IN *A.P.C.A. Abstracts* JULY 1960–SEPTEMBER 1960 (DENOTED BY "A" CODE NUMBER) OF *Public Health Engineering Abstracts* JULY 1960–SEPTEMBER 1960 (DENOTED BY "P").

GENERAL ASPECTS OF AIR POLLUTION

The Anti-Dust Treatment of Coal and Coke Anti staub-Behandlung von Kohle und Koks. W. FRIES. *Gluckauf* (Essen) **95**, 489–501 (April 25, 1959).

A3158/1960

Chronic Bronchitis: An Approach to the Problem of Prevention Together with an Assessment of the Health and Social Cost. J. L. BURN. *Public Health* (London) **74**, 131-8 (Jan. 1960).

A3192/1960

Cancer of the Lung—A Public Health Problem? J. CUTHBERT. *Public Health* (London) **74**, 123–30 (Jan. 1960).

A3193/1960

Polynuclear Aromatic Hydrocarbons in the Particulates of Diesel Exhausts in Railway Tunnels and in Particulates of an Urban Atmosphere. G. E. MOORE and M. KATZ. *Int. J. Air Poll.* (London) **2**, 221–35 (March 1960).

A3194/1960

Electric Space Charges and Human Health. I. H. KORNBLUEH. *Bull. Amer. Meteorol. Soc.* **41**, 361–7 (July 1960).

A3224/1960

Der Fraktionierungseffekt in Thermalprazipitator und die Folgerungen fur ein Elektronenmikroskopisches Kornanalysenverfahren. H. THURMER. *Staub*, **20**, 6–9, (Jan. 1, 1960).

P/1960

SOURCE, POLLUTANTS, CONTROL OF AIR POLLUTION

An Investigation of Carcinogenic Substances in the Effluents from Certain Industrial Processes. Opyl izucheniia kantserogennykh veshchestv stochnykh vodakh nekotorykh proizvodstv. S. N. CHERKIN-SKII, P. P. DIKUM, and G. P. LAKOVLEVA. *Gigiena i Sanit.* (Moscow) **24**, 11–14 (Sept. 1959).

A3159/1960

Facts and Figures Concerning Noxious Substances in the Exhaust Gases of Automobiles and of Combustion Engines in General. Faits et chiffres concernant les substances novices des gaz d'echappement des vehicules automobiles et en general des moteurs a combustion interne. M. SERRUYS. *Rev. pathol. gen. physiol. clin.* Paris, **58**, 239–47. (Feb. 1958).

A3165/1960

Fluorescent Spectra of Aromatic Hydrocarbons Found in Polluted Atmosphere. J. F. THOMAS, B. D. TEBBENS, E. N. SANBORN, and J. M. CRIPPS. *Int. J. Air Poll.* (London) **2**, 210–20 (March 1960).

A3167/1960

Particle Dispersion from Stacks. Stofverspreiding uit schoorstenen. J. P. EHRENBURG and G. B. SMIT. *Electro-Tech.* (The Hague) **37**, 359–68 (Aug. 6, 1959).

A3196/1960

Fundamental Studies on the Thermal Precipitator. Grundlegende Untersuchungen zum Thermalpräzipitator. K. SCHMITT. *Staub* (Düsseldorf) **19**, 416–21 (Dec. 1959).

A3202/1960

Sulfur Dioxide Sensitized Photochemical Oxidation of Hydrocarbons. H. S. JOHNSTON and K. DEV JAIN. *Science* **131**, 1523–4 (May 20, 1960).

A3228/1960

Oxides of Sulfur in the Flue Gases of a Coke-Fired Domestic Boiler. D. MARSDEN and R. A. MOTT. *J. Inst. Fuel* (London) **33**, 267–70 (June 1960).

A3230/1960

Catalytic Oxidation of Hydrocarbons. K. C. STEIN, J. J. FEENAN, G. P. THOMPSON, J. F. SCHULTZ, L. J. E. HOFER, and R. B. ANDERSON. *Ind. Eng. Chem.* **52,** 671–4 (Aug. 1960).
A3234/1960

Reduction of Air Pollution and Control of Emission from Automotive Crankcases. P. A. BENNETT, M. W. JACKSON, C. K. MURPHY, and R. A. RANDALL. *S.A.E. Annual Meeting, Preprint 142A,* 30 pp. (Jan. 11–15, 1960).
P/1960

The Areas Within Concentration Isopleths Down-wind of Continuous Point Source. WILLIAM P. ELLIOTT. *Int. J. Air Poll.* **2,** No. 2, 115–26 (Oct. 1959).
P/1960

Decontamination of Dissolver Vent Gases at Hanford. A. G. BLASEWITZ, R. V. CARLISLE, B. F. JUDSON, M. F. KATZER, E. F. KURTZ, W. C. SCHMIDT, and B. WEIDENBAUM. *HW–20332,* 17 pp. (February 16, 1961). Decl. (July 6, 1959). Nuclear Science Abstracts, 14, 1165 (Jan. 15, 1960).
P/1960

Eliminate Truck Fumes. *Safety Maintenance,* 118, 45–46 (November 1959). Industrial Hygiene Digest, **23,** 1283 (Dec. 1959).
P/1960

ATMOSPHERIC PHENOMENA, DISPERSION, EMISSION

The Travel and Diffusion of the Radioactive Material Emitted during the Windscale Accident. J. CRABTREE. *Quart. J. Roy. Meteorol. Soc.* (London) **85,** 362–70 (Oct. 1959).
A3168/1960

A Synoptic Study of Day-to-Day Changes of Ozone over the British Isles. D. W. MARTIN and A. W. BREWER. *Quart. J. Roy. Meteorol. Soc.* (London) **85,** 393–403 (Oct. 1959).
A3169/1960

Adhesion of Solid Particles to Solid Surfaces. MARGARET C. KORDECKI and C. ORR, JR. *Arch. Environmental Health* 1, 1–9 (July 1960).
A3235/1960

Determination of Particle Size Distribution of Dusts by the Pipette Method. (Ein Beitrag zur Bestimmung der Kornungskennlinein von Stauben mit Hilfe der Pipettemethode). LESZEK KUNCEWICZ and ZDZISLAW KRZYEWSKI. *Staub,* **20,** 47–48 (Feb. 1, 1960).
P/1960

ANALYSIS, DETECTION, MEASUREMENT, MEASUREMENT EQUIPMENT, MEASUREMENT METHOD, RESEARCH, SAMPLING, TECHNIQUES, TESTING

Aromatic Polycyclic Hydrocarbons in Polluted Air as Indicators of Carcinogenic Hazards. H. L. FALK, P. KOTIN and ADELE MILLER. *Int. J. Air Poll.* (London) **2,** 201–9 (Mar. 1960).
A3174/1960

The Drag on Spheres and Cylinders in a Stream of Dust-Laden Air. T. GILLESPIE and A. W. GUNTER. *J. Appl. Mech. Ser.* E. **26,** 584–6 (Dec. 1959).
A3175/1960

The Chromatography of Polycyclic Hydrocarbons. L. DUBOIS, ANN CORKERY and J. J. MONKMAN. *Int. J. Air Poll.* (London) **2,** 236–52 (March 1960).
A3204/1960

An Airborne Cloud Drop Size Distribution Meter. D. P. KEILY and S. G. MILLEN. *J. Meteorol.* **17,** 349–56 (June 1960).
A3205/1960

Polarographic Properties of Smog Aerosols. H. MAYRSOHN and P. P. MADER. *Int. J. Air Poll.* **2,** 283–90 (March 1960).
A3207/1960

An Improved Sedimentometer. Usovershenstvo vannyo sedimentometr. V. I. PANOV. *Ind. Lab.* **24,** 1410–11 (Nov. 1959). Transl. of: *Zauod-Skaaia Lab.* (Moscow) **24,** (10) 1277–8 (1958).
A3208/1960

A Study of the Problem of Particle-Size Measurement. Eine Studie über das Problem der Korngrössenmessung. H. E. ROSE. *Chem. Ingr. Tech.* (Frankfurt) **31,** 183–91 (March 1959).
A3209/1960

The Detection and Determination of Polynuclear Hydrocarbons in Urban Airborne Particulates—I. The Benzopyrene Fraction. E. SAWICKI, W. ELBERT, T. W. STANLEY, T. R. HAUSER and F. T. FOX. *Int. J. Air Poll.* (London) **2,** 273–82 (March 1960).
A3210/1960

The Carbon-14 Content of Urban Airborne Particulate Matter. J. P. LODGE, JR., G. S. BIEN and H. E. SUESS. *Int. J. Air Poll.* (London) **2,** 309–12 (June 1960).
A3240/1960

VOL. 5 1961

The Estimation of Ground Level Concentration from an Elevated Source. P. J. MEADE. *Int. J. Air Poll.* (London) **2,** 303–8 (June 1960).
A3241/1960

Estimation of the Electrical Charge of Dust Particles. Merainie elektrickeho naboja prachovych castic. F. MINARIK, A. STANKOVICOVA, and L. URICEK. Pravcovni lekarstvi (Prague) **12,** (4) 180–5 (1960).
A3242/1960

Aerosol Formation from Gaseous Air Pollutants. M. J. PRAGER, E. R. STEPHENS, and W. E. SCOTT. *Ind. Eng. Chem.* **52,** 521–4 (June 1960).
A3243/1960

Current Applications of the Reverse-Jet Filter Principle. KNOWLTON J. CAPLAN. *A.M.A. Archives of Industrial Health.* **21,** 200–08 (March 1960).
P/1960

Automatic Recording Instruments as Applied to Air Analysis. PAUL M. GIEVER and WARREN A. COOK. *A.M.A. Archives of Industrial Health.* **21,** 233–49 (March 1960).
P/1960

Treatment of Exhaust Gases from Boron Fuels. WALTER H. WILLIAMSON and MAURICE F. CLAY. *American Industrial Hygiene Association Journal,* **21,** 110–14 (April 1960).
P/1960

Ultraviolet, Visible and Fluorescence Spectral Analysis of Polynuclear Hydrocarbons. E. SAWICKI, T. R. HAUSER, and T. W. STANLEY. *Int. J. Air Poll.* (London) **2,** 253–72 (March 1960).
A3211/1960

The Re-Entraiment of Settled Dusts. R. A. BAGNOLD. *Int. J. Air Poll.* (London) **2,** 357–63 (June 1960).
A3236/1960

Geometric Mean Particle Size and Its Application in the Sampling of Airborne Dust. S. GURUSWAMY and V. S. NARASIMHACHAR. *J. Sci. Ind. Research* (New Delhi) **18B,** 319–23 (Aug. 1959).
A3238/1960

Correction for Anisokinetic Sampling of Gas-Borne Dust Particles. S. BADZIOCH. *Journal of the Institute of Fuel,* **33,** 106–10 (March 1960).
P/1960

A Routine Sampler for Detecting Variations in the Emission of Dust and Grit. B. LEES and M. C. MORLEY. *Journal of the Institute of Fuel.* **33,** 90–95 (Feb. 1960).
P/1960

On Horizontal Flat-Plate Sampling of Solid Particles in the Atmosphere. K. D. HAGE, C. H. H. DIEHL and M. G. DUDLEY. *A.M.A. Archives of Industrial Health,* **21,** 124–31 (Feb. 1960).
P/1960

Multi-Channel Photoelectric Scanning Instrument for Sizing Microscopic. B. B. MORGAN and E. W. MEYER. *Journal of Scientific Instruments,* **36,** 492–501 (December 1959).
P/1960

AREA MEASUREMENT, SURVEYS

Bronchitis Mortality Rates in England and Wales and in Denmark. O. W. CHRISTENSEN and C. H. WOOD. *Brit. Med. J.* (London) No. 5071, 620–2 (March 15, 1958).
A3179/1960

BOOKS, MANUALS, SELECTED PUBLICATIONS, SPECIFICATIONS, STANDARDS, TECHNICAL REPORTS

Data for the Estimation of Sanitary-Protective Zones for Electric Power Stations Which Use Peat. Materialk k obosnovaniiu sanitarno-zachchitnykh dlia elektrostantsii, rabotaiushchikh na torfe. N. IA. IANYSHEVA. *Gigiena i Sanit.* (Moscow) **24,** 6–11 (Sept. 1959).
A3183/1960

Air Pollution from Commercial Jet Aircraft in Los Angeles County. R. E. GEORGE and R. M. BURLIN. Air Poll. Control District, Los Angeles, Calif. 1960, $3.50. 61 pp.
A3215/1960

A Study of the Wind Profile in the Lowest 400 Feet of the Atmosphere. I. A. SINGER. Progress Rept. No. 5, Oct. 16, 1959-Jan. 15, 1960. Upton, N. Y. 1960. $1.50. 50 pp.
A3216/1960

ASTM Standards on Methods of Atmospheric Sampling and Analysis. ASTM Committee D-22. American Society for Testing Materials. Philadelphia, Pa. 1959. $2.25. Members $1.80. 104 pp.

A3217/1960

Report No. 30. Sixth Technical Progress Report. W. L. FAITH and N. A. RENZETTI. *Air Pollution Foundation, San Marino, California*, 81 pp. (March 1960).

P/1960

Newly Revised Edition of "VDI-Standards for Particle-Size Determination of Industrial Dusts". (Neue Unerarbeitete Auflage der "VDI-Richtlinien Feinheitsbestimmungen an Technischen Stauben"). Roland Nagel. *Staub* **20**, 109–12 (April 1, 1960).

P/1960

Concepts in Fabric Air Filtration. DAVID G. STEPHAN, GEORGE W. WALSH and ROBERT A. HERRICK. *American Industrial Hygiene Association Journal*, **21**, 1–14 (Feb. 1960).

P/1960

EFFECTS OF AIR POLLUTION

Respiratory and Cardiac Deaths in Los Angeles Smogs during 1956, 1957 and 1958. C. A. MILLS. *The American Journal of the Medical Sciences*, **239**, 307–15 (March 1960).

P/1960

Motor Exhaust Gases and Lung Cancer in Cincinnati. C. A. MILLS. *The American Journal of the Medical Sciences*, **239**, 316–19 (March 1960).

P/1960

VOL.
5
1961

Int. J. Air Wat. Poll. Pergamon Press 1961. Vol. 5, No. 1, pp. 84–85. Printed in Great Britain.

BOOK REVIEWS

Air Pollution: World Health Organization Monograph No. 46. Columbia University Press, New York. 442 pp., $10·00.

THIS monograph, presenting a group of papers prepared for the World Health Organization in 1957, is an important historical document. Its fourteen chapters are prepared by outstanding men in the field of air pollution. It is unfortunate, however, that it could not have come to press more rapidly, since it has now become almost entirely a historical document. The past four years have seen a great deal of progress in the understanding of air pollution, and there is little sign that any attempt has been made to update the papers. Only a handful of references more recent than 1957 were found in the book.

Needless to say, all aspects of air pollution do not progress at the same rate, and for that reason some portions of the volume seem more dated than others. For example, the historical review of atmospheric pollution by E. C. HOLLIDAY suffered only slightly from the passage of four years, since one is accustomed to seeing studies of trends lagging considerably behind the current date. Furthermore, there have been no particularly newsworthy air pollution episodes in the past four years. The same cannot be said for MORRIS KATZ's discussion of some aspects of the physical and chemical nature of air pollution. There has been remarkable progress in the understanding of the physics and chemistry of substances in the atmosphere, so that some of the conclusions drawn in 1957 are now believed to be totally invalid.

From the reviewer's point of view, the chapter by S. CAMBI on air pollution sampling, analysis and instrumentation was particularly disappointing. The author gives a reasonable, if somewhat overly detailed, qualitative discussion of a philosophy of sampling, analysis and instrumentation. However, he stops at this point. No concrete facts are given, no actual analytical methods are mentioned, and there is no bibliography. The paper also seems to fit poorly with several of the other papers. For example, M. D. THOMAS mentions determinations of the phytotoxic effects of concentrations of fluoride below 1 part per thousand million, while Professor CAMBI gives the distinct impression that it takes enormous skill to determine this pollutant at levels as low as 0·1 part per million. He also seems to give the impression that concentrations around this level are the lowest that are of any significance to measure. Since presumably this paper was originally translated from the Italian, one wonders to what extent some of its awkwardness may have been injected by the translator.

These chapters were chosen at random, and should not be taken as exemplifying either the best or the worst in this monograph. The format is uneven; two of the papers have no bibliographical references whatever, and the others vary from footnotes to bibliographies in several different styles. It is unfortunate that, during the long delay in publication, a more uniform job of editing could not be done.

Two compensating errors in algebra are noted on page 121; no other serious typographical errors were noted. The usual semantic confusion caused by the use of the word "smog" to denote two very different sets of atmospheric pollutants cropped up again and again in the book. It is epitomized on page 233 in the statement "There are three principal air pollutants of major interest ... sulfur dioxide, fluorine compounds, and smog ... There are at least two distinct types of smog ..." It seems almost worthwhile to declare a total moratorium on the use of that term, pending formation of a committee on nomenclature to clarify its use.

Despite the above complaints, the book has a reasonably good index, and should be quite useful. The graphic work is excellent, especially the colour plates of damaged plants in the chapter by M. D. Thomas.

JAMES P. LODGE

Atmospheric emissions from Petroleum Refineries. U.S. Department of Health, Education and Welfare

THE compilers of this booklet are to be congratulated on compressing into so small a space such a full and detailed account of the complexities of modern petroleum refining and on including so informative a statement of the many potential atmospheric emissions together with suggestions for assessing their magnitude and for technical methods of abating them.

With that said it may seem an uncharitable paradox to observe that there are some notable omissions. Except for a cursory listing on page 45, there is no mention of the use of CO boilers to burn the carbon monoxide present in catalytic cracking regeneration gases. There is no mention of the modern practice of substituting surface condensers for barometric ones in the ejector systems of vacuum distillation units which greatly reduces both atmospheric and water pollution problems. Hydrogen sulphide is omitted in the text on page 15 and in table 2 from the list of possible emissions, and there is no mention of the modern multi-jet smokeless flare. (The use of steam for smoke suppression is excessively expensive). Finally it is a little surprising that there is no mention whatever of the modern Petrochemicals industry.

Readers of this booklet would do well to bear in mind that it has been compiled from a survey of refineries in the United States and of refining practice in that country. Thus the figures given on yields from crude, plant capacities, etc. apply to the United States only and are not necessarily representative of refining practice elsewhere.

Furthermore there is some degree of over-emphasis on the importance of hydrocarbon losses. This is undoubtedly because of the smog problem in the Los Angeles area and the work that has been done on this problem by the Los Angeles County Air Pollution Control District. It must, nevertheless, be realized that this smog problem is unique to the Los Angeles area and the conditions that favour smog formation through the mechanism of the Haagen-Smit reaction occur hardly anywhere else in the world.

Provided these caveats are borne in mind, the booklet is of considerable value.

A. W. W. KIRBY

FILMSTRIP REVIEW

Chronic Bronchits—A medico-social problem. Filmstrip and lecture notes produced by Diana Wyllie Limited, with the co-operation of the Medical Officer of Health, City of Salford, and Pfizer Limited.

VOL. 5 1961

THIS filmstrip is well produced and should be of value to Health Departments and to others presenting the importance of clean air and the general prevention of chronic bronchitis.

The first section deals with the incidence and causes of chronic bronchitis and shows vividly the effects on the individual of this disease. It also presents graphically some statistics of the effects of the disease on community health. It is rather unfortunate that these graphic representations do not seem to come out too clearly and probably more vivid colours would have been helpful.

The second section deals with the general treatment and management of patients suffering from chronic bronchitis, whilst the third section presents the public health aspects of the subject.

This filmstrip should be invaluable for presenting the importance of clean air to lay audiences and will certainly give the lecturer an opportunity to amplify every aspect of the subject.

ANDREW B. SEMPLE

PAPERS TO BE PUBLISHED IN FUTURE ISSUES

A. D. Osborne, E. F. Darley and J. N. Pitts Jr. : On the stability of acrolein towards photo-oxidation in the near ultra-violet.

R. E. McKinney and G. J. Mohanrao : A study of the biochemical characteristics of quarternary carbon compounds.

Perry L. McCarty and Clement A. Vath : Volatile acid digestion at high loading rates.

F. Wipperman and W. Klug : Ein Verfahren zur bestimmung von Schornsteinmindesthöhen.

A. P. Altshuller, A. F. Wartburg, I. R. Cohen and S. F. Sleva : Storage of vapours and gases in plastic bags.

E. N. Lawrence : Atmospheric pollution (sulphur dioxide) in hilly terrain.

gaseous components by Orsat analyses or other similar chemical techniques are prohibitively time consuming and often do not possess the sensitivity required for the small volume of gas sample resulting during a short interval sampling program.

The chromatographic method can reliably analyze a minimum volume of 1 ml of a gas in a total time of 5 to 10 min depending on the components desired. This small sample size permits the use of conveniently small bench-top experiments.

CALIBRATION

The characteristic steep narrow-peaked curves produced by the recorder stimulate the imagination as to the geometrical techniques possible for evaluating them. The curves plotted by the recorder show the relation of component gas concentration in the carrier gas to column flushing time. Therefore the area under the curve being a product of these two parameters is the volume of component gas. Consequently measurement of the area under the curve by means of a planimeter was selected for the work reported here. Due to the similarity in shape of these curves it can be argued that the product of peak height and base width, peak height alone, or some other relationship would give a suitable area value.

Because the range of the recorder is limited, and because any component gas concentration may be found, the instrument has a sensitivity control (attenuation) in the electrical circuitry to permit adjustment of the height of the recorded curve and thereby, necessarily, the area under it. Fig. 3 shows three curves produced by a 10 ml sample of carbon dioxide, with relative recorder attenuations of 50, 100, and 200. It will be noted that when the actual areas are multiplied by their respective attenuation values, the component areas are the same because they all represent 10 ml samples.

VOL. 5 1961

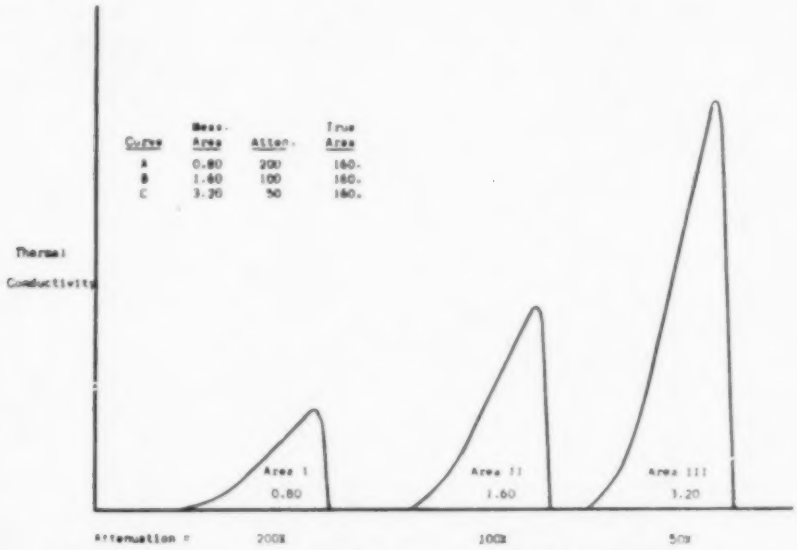


FIG. 3. Recorder sensitivity effects. Gas sample = 10 ml carbon dioxide.

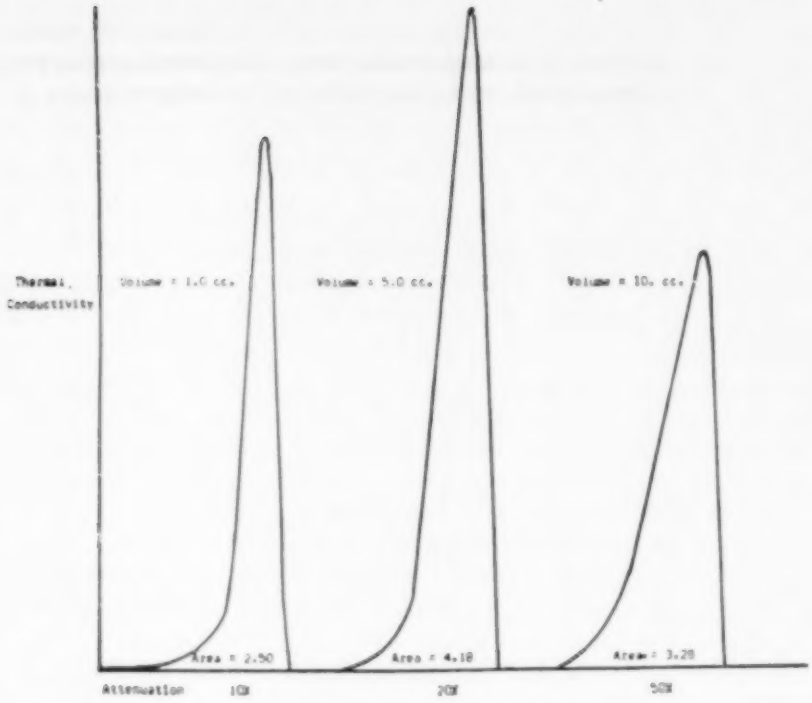


FIG. 4. Chromatograph calibration. Carbon dioxide (30 lb/in², 40°C). Silica gel column.

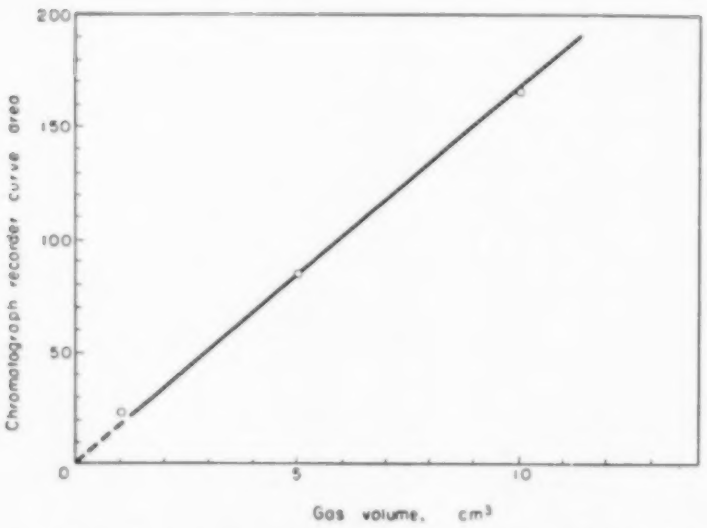


FIG. 5. Carbon dioxide calibration curve. Silica gel column. (30 lb/in², 40°C).

In a typical calibration, as shown in Fig. 4, the gas volumes of carbon dioxide were 1·0 ml, 5 ml, and 10 ml, but the curve areas showed no such similarity in area because it was necessary to utilize a sensitivity value in each case which gave the maximum measurable area on the chart paper. However, it was the computed area which was used to establish the actual calibration curve shown in Fig. 5. It will be noted that the calibration curve appears to pass through the origin of the axes as expected, but, because of the sample's size, it does not appear advisable to use this technique routinely to calibrate for gas volumes less than 1 ml. Quite often small concentrations of a desired gas component may occur as a normal respiratory action. Verification of the calibration curve in the smaller sample size range can be achieved by using known dilutions of the sample gas in an inert gas (i.e. helium).

The sensitivity of the instrument will permit evaluation of component gas volumes as small as 0·01 ml, or 0·03 mg, by assuming curve areas as small as 0·25 in^2 can be measured. This technique would yield a more reliable calibration curve for gas volumes between 1 ml and zero.

BIO-OXIDATION SUBSTRATE METABOLISM

For the illustrative metabolism described here, an activated sludge culture (approx. 3000 mg/l.), was developed in a 3 l. aeration vessel. The unit was fed daily with a concentrated skim milk solution, and prior to feeding, 1 l. of supernatant liquor was decanted and this volume was replaced with tap water. The surplus air and escaping respiratory gases were conducted through the gas sampling valve of the chromatograph from which they discharged through a flow-rate meter.

After the aeration flow rate was established at a satisfactory and uniform value

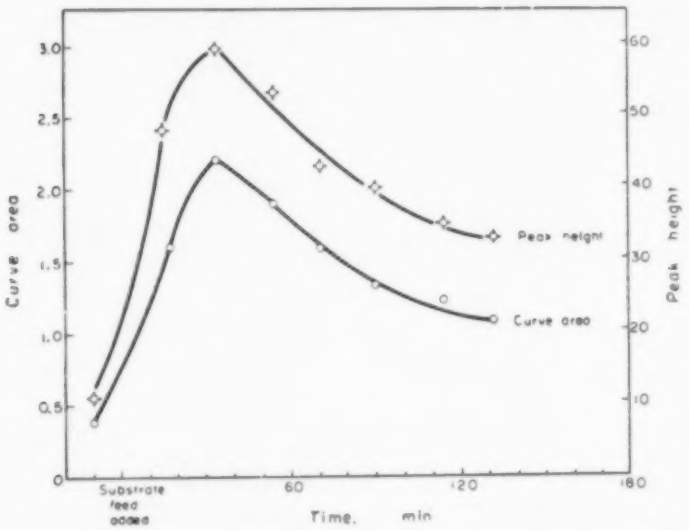


FIG. 6. Activated sludge substrate metabolism. Simulated chromatograph curves carbon dioxide (sample volumes = 10 ml).

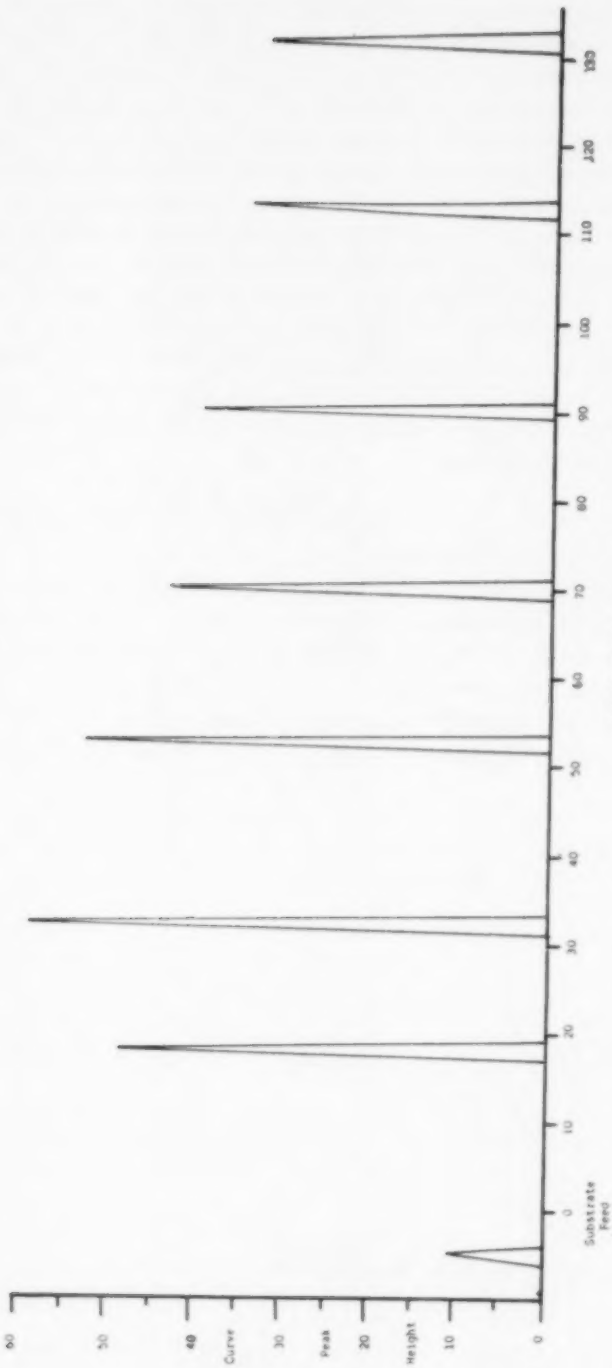


FIG. 7. Carbon dioxide respiration. Activated sludge substrate metabolism.

several samples were analyzed for carbon dioxide content to obtain a "background" or reference walue of the unfed activated sludge. One of the reference peaks is portrayed at the left in Fig. 6. Then the skim milk solution was introduced into the substrate of the aeration vessel, after which the exhaust gas was sampled for carbon dioxide at the intervals shown in Fig. 6. In order to portray the trend in the recorded curves and the sampling intervals as the substrate metabolism proceeded, the curves in Fig. 6 are graphic portrayals of the actual curves. In Fig. 7 the data concerning the true curve areas are plotted vs. metabolism time, as indicated. For comparison, the peak-height values vs. metabolism time are also plotted in Fig. 7. Only the early portion of the complete substrate metabolism is shown in Figs. 6 and 7, although the curve in Fig. 7 returned to the original reference value in 12–18 hr.

Using this technique it is possible to integrate the area under the "Curve Area" plot in Fig. 7 and evaluate the total carbon dioxide respired from a particular substrate. Without resorting to the area under the "Curve Area" plot, the rate of metabolism can be determined from the ordinate values. If metabolism rate is the only value desired then as can be seen in Fig. 7 this can be determined from either of the curves. It appears possible, if desired, to be able to calibrate the chromatograph in terms of peak heights of the curves.

While not illustrated here, the unused oxygen also is determined during this same analysis, but prior to the carbon dioxide determination. By reducing the input rate of oxygen to the aerator to a minimum metabolic value it would be possible to observe the oxygen uptake corresponding to the carbon dioxide respiration but it is necessary to keep the activated sludge continually suspended. Such minimum oxygen flow rates will not maintain suspension of the sludge and it will be necessary to employ some auxiliary technique for this purpose.

By further reducing the oxygen supplied to a system to zero the metabolism could be forced to anaerobic conditions while chromatographically recording the sparged gases during the change. Then, assumedly, the system could be restored to its original aerobic conditions.

VOL. 5 1961

ANAEROBIC METABOLISM

No examples of anaerobic metabolism are presented here as they would be repetitive of much data which have been published. Chromatographic analytical control of the digester gas composition correlated with the rates and compositions of feed to the digester during sewage plant operation might reveal valuable data for improving digestion efficiency and benefits. It is probable that once the biomechanics of the metabolism is fully understood a higher "high-rate" digestion process might be possible.

Acknowledgements—The authors are very indebted to A. J. McDonnell, Graduate Assistant in Sanitary Engineering, and to George Johnstone, Sanitary Engineering undergraduate, for their sincere perseverance and assistance that made this report possible.

AEROBIC SLUDGE DIGESTION

N. Jaworski, G. W. Lawton and G. A. Rohlich

University of Wisconsin

INTRODUCTION

Most of the experimental reports on the aerobic digestion of domestic sewage sludges have been on mixed liquors with suspended solids concentrations of less than 10,000 ppm. Some work has been done on aerobic digestion (fermentation) of sludges at higher concentrations.

In 1950 Coackley (1, 2) carried out a series of experiments to determine the extent of aerobic digestion of sludge previously subjected to anaerobic digestion. In one series, the anaerobically digested sludge was aerated without inoculation by aerobic organisms. Another series was inoculated with aerobic organisms found in a compost heap. The third series continued the anaerobic digestion of the sludge. The three series were carried on up to 71 days of digestion with temperatures of 18°C and 37°C.

VOL. 5 1961

Although Coackley was primarily interested in dewatering characteristics of the sludge, he also observed the reduction of solids and organic nitrogen. At a temperature of 18°C, the reduction of volatile solids after 48 days of digestion was not appreciable, even in the inoculated series. In the 37°C non-inoculated series, the volatile solids were reduced from 2·98 per cent to 1·08 per cent after 47 days, while in the inoculated series, they were reduced from 2·66 per cent to 1·52 per cent. A decrease in organic nitrogen was also noted in the aerobic units which did not occur in the anaerobic series. The aerobic series produced a stable sludge which showed no signs of decomposition when left without aeration. Anaerobic digestion could not be initiated in this series even after the digesters were inoculated with anaerobic organisms.

Murphy in 1959 (3) studied the effects of aeration on the filterability and the settleability of sewage sludges. He used primary and waste-activated sludges, mixed at a ratio of 1 : 1 by volume, and aerated the sludges for various lengths of time at controlled temperatures. Most of his work was done at 15°C. He concluded that the reduction of volatile solids at 15°C with digestion times up to 6 days was not appreciable.

Akers (4) of Iowa University investigated some of the factors that may influence the auto-oxidation (endogenous respiration) rate of biological sludge. Two primary factors were considered in his study: (1) the effect of auto-oxidation rate on the mean sludge age (pounds of suspended volatile solids in the system divided by the pounds of suspended volatile solids into the system daily); and (2) the effect

of separate aeration (aerobic digestion) of the mixed liquor without additional substrate being added during the aeration period.

In his study the average reduction in volatile solids in 8 days at 23°C was greater than 20 per cent. The corresponding reductions in suspended volatile solids, B.O.D. and C.O.D. were 30 per cent, 60 per cent and 35 per cent, respectively.

His study also indicated that the rate of change of auto-oxidation is considerably less than the rate of change of the sludge age.

Eckenfelder (5) studied aerobic digestion using waste activated sludge from a conventional activated sludge plant. Four-liter digesters were aerated for a period of 7 days with the temperature maintained at 25°C. At the end of the 7-day period of aeration, his data showed a reduction in C.O.D. of 48·5 per cent, a reduction in suspended volatile solids of 38·2 per cent, and a decrease in volatile solids content from 76·5 per cent to 63·5 per cent. The soluble nitrogen increased from 46 ppm to 94 ppm in the 7 days of aeration.

The work reported herein was undertaken to study the effects of temperature, time and loading rates on the aerobic fermentation of blends of primary and waste activated sludges. Both batch-type and incremental feeding (referred to as continuous feeding) studies were made. The sludges used were obtained from the east section of the Nine Springs Sewage Treatment Plant, at Madison, Wisconsin. This sludge is produced from a domestic waste comprised of approximately one-third pretreated meat packing waste and two-thirds domestic sewage.

EXPERIMENTAL APPARATUS

The digesters used in this study were 6 l. Erlenmeyer flasks with 4 in. diameter holes in the base. In operation, the flasks were inverted and placed in steel drums which served as constant temperature baths.

Air was supplied to the bottom of each digester by a glass tube inserted through a rubber stopper. These tubes produced air bubbles large enough to keep the digesters well-mixed in addition to supplying oxygen for the microbial population. The rate of air supplied to the digesters was measured by calibrated orifices and rated water manometers mounted on a nearby panel.

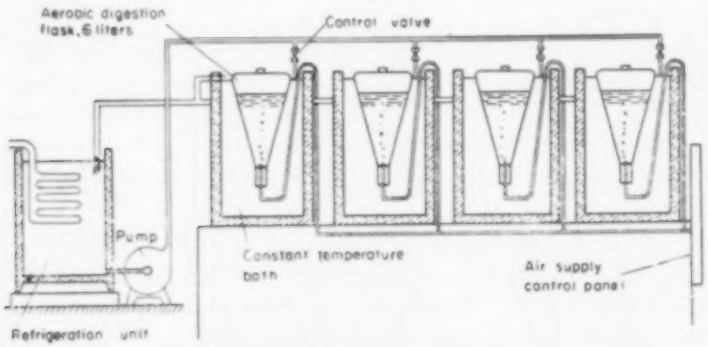


FIG. 1. Sketch of aeration apparatus.

The temperature was controlled within ±0·5°C separately in each bath by means of a 300 W heating coil regulated by a precise temperature sensing element and relay. Cooling water from a refrigerated source was pumped into the baths by means of a ½-in. centrifugal pump and returned to the refrigerated tank by gravity. The flow to each bath was regulated separately, thus permitting more precise temperature control. A sketch of the aeration apparatus is shown in Fig. 1.

EXPERIMENTAL PROCEDURE

Batch digestion studies

Batch runs were made in which temperature, digestion time, and solids loading were varied. A starting volume of 4 l. of substrate was used in each of the digesters with 0·20 ft³/min of air being supplied throughout the batch runs. Evaporation losses were made up with distilled water prior to daily sampling. The substrates were blends of primary and waste activated sludge.

Preliminary studies on a batch basis indicated that solids concentration and temperature were important variables. Aeration of sludges which had total solids concentrations of 0·75 and 3·06 per cent for 6 days at 20°C gave C.O.D. reductions of 50 and 12 per cent, respectively. At 0·75 per cent solids the reduction in volatile matter in 6 days ranged from 28 to 44 per cent for temperatures of 15° and 25°C. At a solids concentration of 3·06 per cent reductions in volatile matter were considerably lower, being only 11 per cent at 15°C.

VOL. 5 1961

Continuous digestion studies

The feed used in the continuous digestion studies contained approximately 3·2 per cent total solids of which from 70 to 80 per cent were volatile matter. This feed consisted of a 1·75 to 1·00 ratio (dry solids basis) of primary and waste-

TABLE 1

Detention time days	Temperature degrees C	Volume of sample ml	Loading rate volatile solids #/ft³/day
5	15	800	0·290
5	20	800	0·290
5	35	800	0·290
10	15	400	0·145
10	20	400	0·145
10	35	400	0·145
15	20	267	0·096
30	15	133	0·048
30	20	133	0·048
60	15	67	0·024
60	20	67	0·024

activated sludges. The air rate was maintained at 0·20 ft³/min per digester. The digesting volume was held at 4·00 l. throughout the study with evaporation losses being made up daily prior to sampling and feeding.

The detention time controlled the size of daily samples and loading rate. For example, with a 4000 ml working volume and a 10 day detention time, a 400 ml sample was withdrawn daily and the same volume of feed material added. With the feed being held constant at 3·2 per cent total solids (72·5 per cent volatile matter) the 10 day detention time fixed the volatile solids loading rate at 0·145 lb per ft³ of digester capacity per day.

Table 1 shows the detention times, sample sizes, loading rates, and temperatures used in the continuous digestion studies.

In the continuous feeding studies, digestion was carried on until equilibrium conditions were attained before analyses were made on a routine basis. The digestion was then continued for approximately 1 month with analytical data being obtained at intervals of from 1 to 3 days. The data used in this report thus represent equilibrium conditions for each detention period considered. A total of eleven series of continuous aerobic digestions was conducted.

The effect of temperature and time of digestion on the volatile solids content of aerated sludge is shown in Fig. 2. The per cent reduction curves indicate that

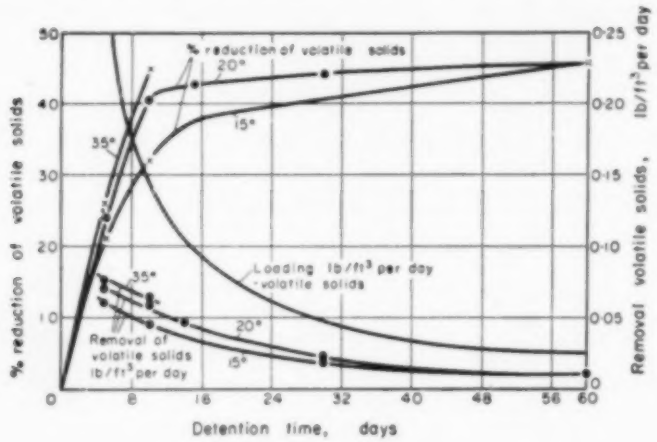


FIG. 2. Aerobically digested sludge.

sharp increases in volatile solids removal take place as the detention time is extended up to about 12 days. Beyond that time the per cent reduction shows only moderate increases, indicating that for the temperatures considered the practical upper limit of detention time, as far as volatile solids reduction is concerned, is approximately 15 days. It is important to note that the loading rate varies inversely with the detention time in these studies, thus it appears that the maximum practical loading rates, based on per cent reduction in volatile solids, are approximately

0·1 lb of volatile solids per cubic foot per day, again considering only volatile solids reduction.

Considering the volatile solids in terms of pounds removed per cubic foot of digester space per day it appears that short detention periods are definitely more practical. The efficiency of the digester, based on the pounds of volatile solids removal, increases as the loading rate increases, however other factors such as drainability, drying characteristics of the remaining solids, and supernatant characteristics also must be considered. The curves in this figure indicate that while temperature has an appreciable effect at short detention times, this effect decreases considerably as the time is lengthened (or the loading rate decreased). At an 8 day detention time the 35°C digester removed about 40 per cent more volatile solids per cubic foot than did the 15°C digester. On a percentage basis the removals range from 38 to 28 per cent respectively at the two temperatures.

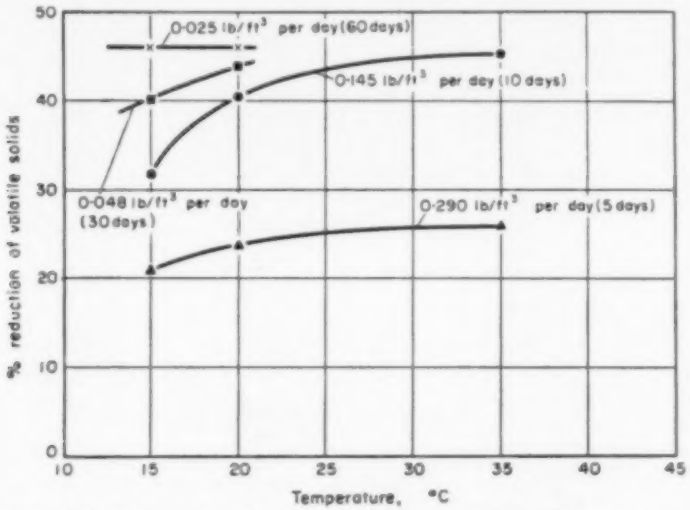


FIG. 3. Aerobically digested sludge. (Loadings refer to volatile solids. Detention time.)

Fig. 3 shows more clearly the effect of temperature on per cent reduction in volatile solids at various loading rates or detention times. For long detention times (60 days) temperature has little effect since digestion is essentially completed at all temperatures. For short detention times (5 days) the effect of temperature also appears to be of minor importance, perhaps because of the heavy loadings to the digester. At intermediate detention times (and loadings) temperature produces more noticeable effects on the volatile solids reduction, with the higher temperatures being more effective. Further studies at various loading rates for fixed detention periods would provide valuable supplementary data.

The effects of temperature and detention time on pH, alkalinity, and concentration of solids are presented in Fig. 4. Solids concentrations, total and volatile, show little change after 15 day detention periods at both 15° and 20°C. Alkalinity

VOL. 5 1961

shows a gradual decrease for detention periods up to about 30 days at the temperatures studied. Alkalinity for the 60 day detention was at approximately the same level as for the 30 day detention period. The pH increased for detention periods up to about 10 days, to a maximum value of about 8·0, and then decreased gradually to values near 5·0. These low pH values found at equilibrium in the long detention times are usually considered to be unsuitable for aerobic metabolism. However, in only one case did there appear to be appreciable build up of volatile solids in the digester after long aeration (65 days) following equilibrium conditions.

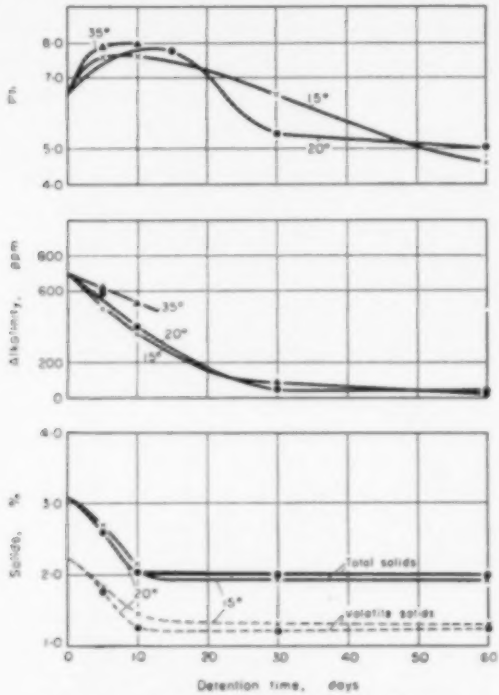


FIG. 4. Aerobically digested sludge (effluent).

The effects of temperature and detention time on the concentration of ammonia, nitrites, and nitrates are presented in Fig. 5. The curves in this figure indicate the usual relationship between ammonia, nitrites, and nitrates in aerobic fermentation. It is noted that more than 600 ppm nitrate-nitrogen was formed at 60 days detention time, indicating that nitrification was proceeding uniformly even at the low pH levels present. Summary data are presented in Table 2.

The drainability of the sludges was measured on gravity sand filters supported by filter paper in Buchner funnels. Filtrate volumes were determined over periods of several hours. It was found that sludges digested for 5 days usually exhibited poorer drainability than undigested sludges, while sludges digested for 10 days or longer showed improved drainability. In general, for detention times of 10 days

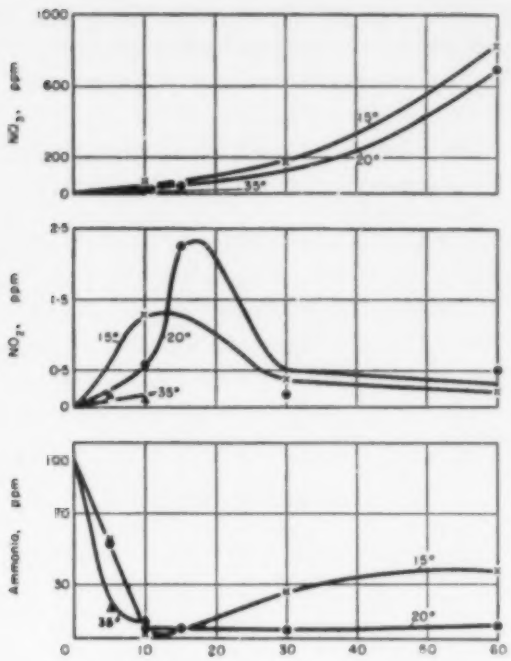


FIG. 5. Aerobically digested sludge (effluent).

or more, from 70 to 80 per cent of the original volume of the sludge appeared as filtrate within 1 hr, as compared to about 40 per cent as filtrate in 1 hr from the untreated sludge. With detention times of 10 days or greater the effect of digestion temperature on drainability appeared to be of minor importance.

TABLE 2. SUMMARY DATA

Detention days	Temp. Cent.	V. solids % Red.	pH	Alkalinity ppm	NH_3 ppm	NO_2 ppm	NO_3 ppm
5	15	21	7·6	510	54	tr	none
5	20	24	7·6	590	54	tr	none
5	35	26	7·9	630	14	0·18	none
10	15	32	7·6	380	3·2	1·28	64
10	20	41	7·6	390	4·9	0·59	60
10	35	45	8·0	540	10·0	0·08	none
15	20	43	7·8	560	7·0	2·27	29
30	15	40·5	6·6	81	4·0	0·36	170
30	20	44	5·4	31	28	0·19	275
60	15	46	4·6	23	38	0·23	835
60	20	46	5·1	35	7·0	0·51	700

When digested sludges were dried for a period of 4 days, cracks appeared in the sludge cakes. No obnoxious odors developed during the drying process. The sludge cake, after 1 week of air drying was low in moisture content and held together firmly.

Few of the digested sludges exhibited good settling characteristics. Sludges digested for short periods of time, and at low temperatures showed poor settleability, perhaps due to the highly dispersed state of the undigested particles. Sludges that had been digested for 30 days formed gas bubbles which caused the solids to float. In some cases gas bubbles appeared in approximately $\frac{1}{2}$ hr, probably the result of denitrification. The gasified sludges would not settle unless the gas bubbles were mechanically removed. Only the sludges digested for 60 days at 15°C and 20°C and those digested for 10 days at 35°C showed improved settling characteristics.

B.O.D. values of the supernatant liquors (Table 3) were less than 100 ppm in all samples analysed with the exception of one liquor from a digester having a 10

TABLE 3. SUPERNATANT CHARACTERISTICS

Detention days	Temperature Centigrade	B.O.D. ppm	C.O.D. ppm
5	35	76	670
10	15	170	—
10	20	55	—
10	35	30	360
15	20	25	440
30	15	78	—
30	20	37	—
60	20	25	460

day detention time. The values ranged from 25 to 78 ppm with the exception of the one value of 170 ppm. C.O.D. values of the supernatant liquors analyzed ranged from 360 to 670 ppm for the various time and temperature conditions studied.

At the end of the final series of the continuous digestion study, the digesters were adjusted to a pH of 7·2 with caustic soda, and aerated without further feeding. The pH was maintained at 7·2 and aeration continued for 47 days. The volatile solids content of the sludges decreased from an average of about 62 per cent down to 55 per cent in those digesters where detention times previously had been 10, 15, and 60 days. In the digester which previously had a five day detention period the volatile solids content decreased from 70 per cent down to 47 per cent. It is likely that the more completely digested sludges had accumulated a greater amount of volatile matter which was more difficult to digest than had the sludge which originated from the five day detention period digester.

Acknowledgement—The work reported herein was supported by a grant from the Walker Process Equipment Company, Aurora, Illinois.

CONCLUSIONS

Data obtained from this study indicate that aerobic digestion of mixtures of raw and activated sludge is feasible. From a consideration of the results obtained it was concluded that:

(1) Reduction of volatile solids is a function of detention time. However only small increases in the reduction are obtained beyond digestion periods of 15 days.

(2) In general greater reductions in volatile solids are obtained at higher temperatures of digestion.

(3) In general higher volatile solids removal is obtained at lower loading rates.

(4) Settling characteristics of sludges digested for 30 days or less are generally poorer than those of the undigested sludges.

(5) Sludges digested for periods greater than 5 days show satisfactory drainability characteristics.

(6) Supernatant liquors from aerobic digesters exhibit relatively low B.O.D. values when compared to anaerobic digester liquors.

(7) Drying of aerobically digested sludges produces little odor.

REFERENCES

1. COACKLEY, F., Research on sewage sludge carried out in the C. E. Dept. of University College London. *J. Inst. Sew. Purif.* 59–72 (1955).
2. COACKLEY, P., Laboratory scale filtration experiments and their application to sewage sludge dewatering. *Biological Treatment of Sewage and Industrial Waste* vol. 2, pp. 287–8. Reinhold, New York (1958).
3. MURPHY, K. L., Sludge conditioning by aeration. Unpublished Master's Thesis, University of Wisconsin (1959).
4. AKERS, W. L., The effect of sludge age on the auto-oxidation rate of long-term aeration of activated sludge. Unpublished Master's Thesis, University of Iowa (1959).
5. ECKENFELDER, W. WESLEY, Studies on the oxidation kinetics of biological sludges. *Sewage and Industrial Waste* **28**, 983–9 (1956).

VOL. 5 1961

DISCUSSION

DOUGLAS E. DREIER: Experience to date indicates use of about 3 ft³ of aerobic digester capacity per capita in the contact stabilization plants allows reasonable storage capacity. Ultimate sludge disposal for most package plants is by tank hauling service, or by sand drying. Since both are costly, the ability to concentrate the sludge is important. These digesters are fed sludge from the re-aeration zone which is already entering the endogenous phase. The feed to the digester is automatically controlled. A decant overflow arrangement from a quiescent settling area in

TABLE 1. AEROBICALLY DIGESTED SLUDGE

pH	5·6
Alkalinity (ppm)	283
Total solids (%, concentrated by decanting)	2·76
Volatile solids, (%)	39·5
Nitrate nitrogen (ppm)	48
Ammonia nitrogen (ppm)	1·75
B.O.D. of supernatant (ppm)	16
Color of supernatant	Yellowish and slightly turbid

the digester automatically concentrates the sludge as new feed is added. Emphasis in these package plants has been to provide for a minimum of operator attention; hence the digester is generously sized, and as automatic in operation as possible.

Table 1 illustrates some of the characteristics of a typical aerobically digested sludge from a contact stabilization plant serving the Garden State Plaza shopping center at Paramus, N.J.*

The waste sludge in this plant had a pH of 7; total solids of 1 per cent, 0·25 ppm of nitrates; and the solids were only 48·8 per cent volatile. A reduction of 32 per cent in volatile solids was obtained in the aerobic digester, even though the feed sludge was already quite low in volatile solids. No visible biological life was found in digested sludge but many dead organisms, mostly *Arcella*, were present. The feed sludge contained high concentrations of protozoan life, good zoogloeal growth, and algal plankton.

While 10–15 days displacement gave almost as good volatile solids reduction as did the 30 and 60 day displacements, for package type plants the longer detentions might still be desirable to provide more sludge storage and reduce the frequency of ultimate disposal. Also, until the detention time was 30 days or more, sludge settleability was not very good; however, it is very possible that the intense aeration required in the shallow container affected settleability results.

The experimental digesters at the University of Wisconsin were operated on the basis of daily feed, with an equal volume of digested sludge at the homogenized concentration withdrawn daily. With this method of operation, most of the organic matter inert to digestion was not accumulated, so the per cent volatile content of solids in the digester did not drop below about 60 per cent. In the package activated sludge plants, the digested solids are accumulated by decant thickening so the percent volatile solids in the digester often is as low as 38–40 per cent.

* *Survey of the Characteristics of Aerobically-Digested Sludge*, Report by Edward R. Grich, Inc., Sanitary Chemists, Paterson, N.J. (1958).

OXYGEN TRANSFER IN THE BIOLOGICAL TREATMENT OF SEWAGE

SHUICHI AIBA, TOYOKAZU YAMADA, AKIRA YAMAMOTO and SUMI SHIMASAKI
University of Tokyo, Japan

VOL. 5 1961

SECTION I

THE mechanism of oxygen transfer from air bubbles to water was studied. A glass vessel 29·8 cm in height and 4 cm square was employed. Dissolved oxygen was measured on a recording polarograph.

Water which contained KCl (0·1 N) and gelatine (0·01 per cent) was first used as a test solution. Malt extract and simulated culture media containing glucose or peptone were then employed to see the effects of organic substances on the oxygen absorption rate. Settled sewage from one of the municipal waste treatment plants, Mikawashima, Tokyo, was also used.

The oxygen transfer coefficient K_LA was determined from the logarithmic plot of saturation deficit vs. aeration time.

The linear correlation between the oxygen transfer coefficient and depth is shown in Fig. 1, the parameter being the air (oxygen) flow rate. It is postulated that K_LA is composed of the following two terms:

$$K_LA = K_LA_0 + K_L'A_0' \tag{1}$$

where K_LA_0 = mass transfer coefficient of bubbles which ascend through a definite liquid depth,

$K_L'A_0'$ = equivalent mass transfer coefficient which is related, if any, to the foaming liquid surface and the bubble formations at the nozzle.

It was assumed that $K_L'A_0'$ could be read from the intersections of each linear correlation, after extrapolated, with the ordinate. The extrapolations are considered to be possible, because no appreciable change of the foaming liquid surface pertaining to each gas flow rate was observed within the experimental range of the depth. K_LA_0 of each experimental condition was then evaluated from equation (1).

In order to determine K_L, the interfacial area A_0 between bubbles and liquid was calculated from equation (2), assuming that bubbles were spherical, d in diameter,

$$A_0 = \pi d^2 n \frac{H}{v_b} = \frac{V_s}{10} \frac{H}{d \cdot v_b}. \tag{2}$$

In equation (2), d, which was measured from the photographs of air bubbles in water, was postulated to be identical in systems other than air and water for the same flow rate of gas. It was observed that the bubbles were usually split into small

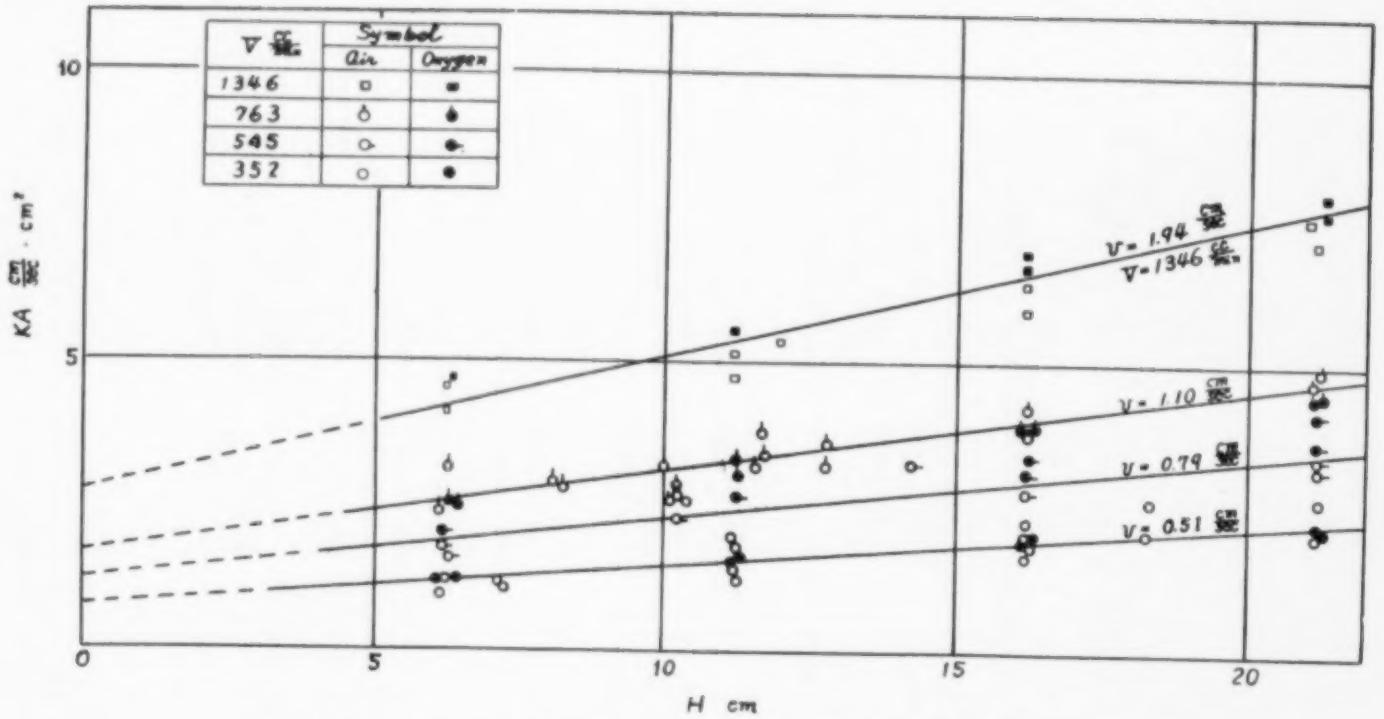


FIG. 1. Correlation between KA and H in the case of air-water and oxygen-water systems.

ones in the experimental range of gas flows. Therefore, small bubbles, $d \leqq 0{\cdot}02$ cm, were excluded in determining d. The maximum error of d which was expected from this approximation was about 10 per cent.

The terminal ascending velocity v_b in equation (2) was estimated using equation (3), which is an empirical correlation presented by van Krevelen *et al.*(6)

$$\frac{\bar{d}g\Delta\rho}{v_b^2\rho} = 2. \tag{3}$$

The published data indicate that the maximum error of v_b in using equation (3) is of $\pm$ 30 per cent. Thus, the maximum error of evaluating A_0 from equation (2) will be of the order of $\pm$ 40 per cent.

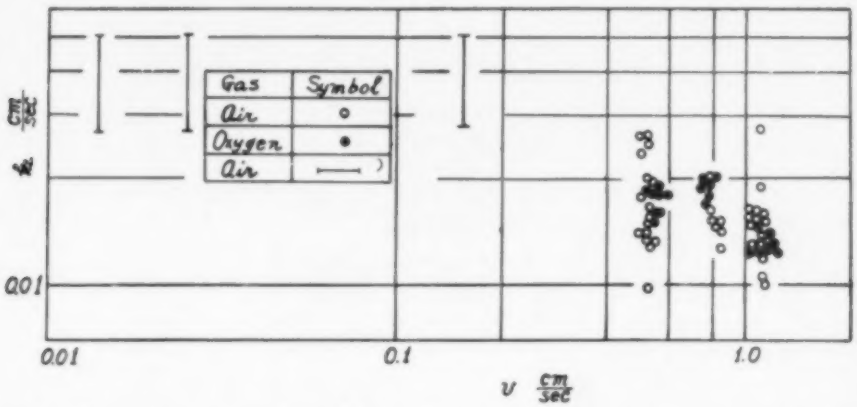


FIG. 2. Correlation between k_L and v in the case of air–water and oxygen–water systems.

Fig. 2 indicates a correlation between k_L and v in the case of air–water and oxygen–water systems. The ordinate is the liquid film coefficient of oxygen transfer k_L, while the abscissa represents a nominal velocity v of gas. In this experimental range of v, the ratio of air flow rate per min to the liquid volume V_a/V_0 was from 1·5 to 19. No dependence of k_L on V_a/V_0 was noted, however.

The mean bubble diameter $\bar{d}$ which had been determined prior to the calculation of these data of k_L was in a range from 0·50 to 0·53 cm. Since the published data of liquid film coefficient of oxygen transfer in aeration are few, only the data presented by Ippen *et al.* (2, 5) are included in Fig. 2. Bubbles corresponding to the data of Ippen *et al.* range from 0·13 to 0·25 cm in diameter.

The difference of k_L in the case of air–water and oxygen–water systems is not apparent. It might be concluded that the controlling resistance against oxygen transfer is found in the liquid film in a range of this experimentation.

The scattering of the data might have resulted from the poor reproducibility of the experiment. A direct comparison between the data of Ippen *et al.* and those of this experimentation is difficult due to the difference of bubble diameters and the fact that the end effects were included in the determination of k_L in the former.

VOL. 5 1961

The liquid film coefficient k_L of a specific sewage and the effects of organic substances on k_L are shown in Fig. 3. The coefficient of the sewage was about the same order of magnitude as that of water.

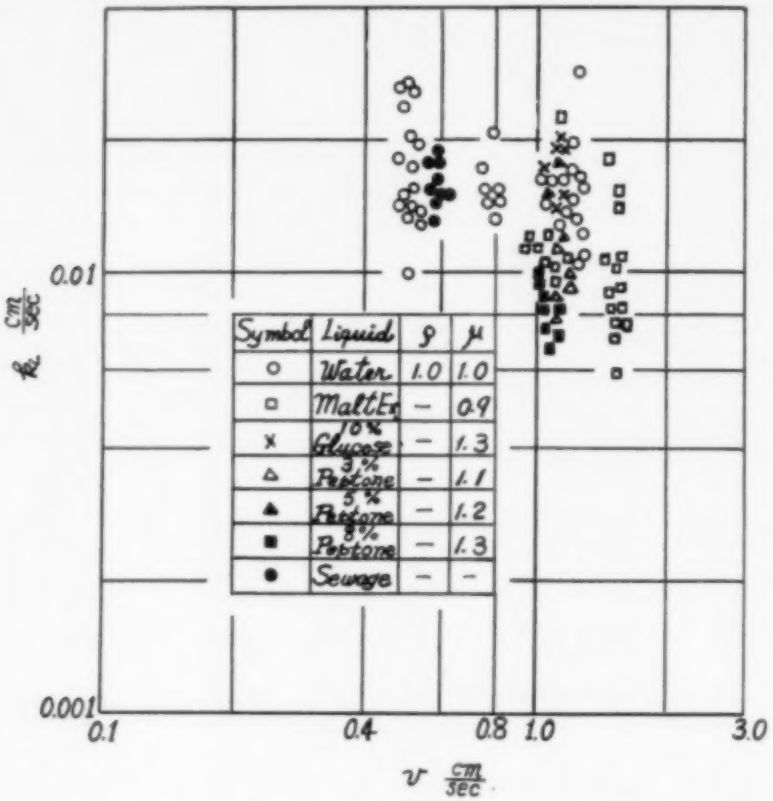


FIG. 3. Effects of organic substances on k_L.

SECTION II

The overall coefficients of oxygen absorption in mixed liquor was determined in an aeration tank of acryl 30 × 30 × 40 cm with a sieve plate containing 225 openings each 0·1 cm in diameter for aeration. The activated sludge was killed by 5 ml of phenol prior to dissolved oxygen measurement.

EXPERIMENTAL RESULTS AND DISCUSSIONS

An example of the experimental results of the respiration rate is shown in Fig. 4(a). It is clear that the respiration rate of the sludge is markedly decreased during a short period of contact time (t) (4). The same trend was also noted in other runs of these experiments.

The respiration rate was expressed approximately as follows:

$$r_r = at + b, \qquad 0 \leqq t \leqq t_1, \tag{4}$$

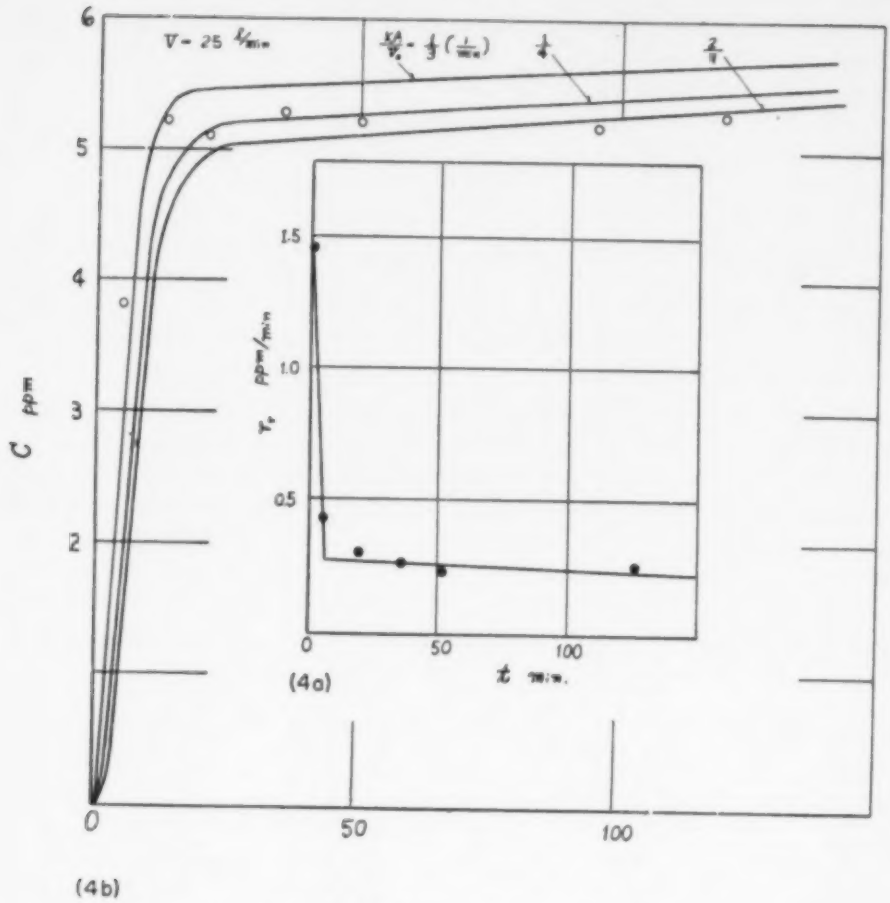


FIG. 4. (a). Respiration rate vs. aeration time.
(b). Estimation of KA/V_0.

$$r_s = a't+b', \qquad t_1 \leqq t. \tag{5}$$

The rate equations (6) and (7) correspond to the respirations of (4) and (5), respectively.

$$\frac{dC_1}{dt} = \frac{K_L A}{V_0}(C_s - C_1) - (at+b), \tag{6}$$

$$\frac{dC_2}{dt} = \frac{K_L A}{V_0}(C_s - C_2) - (a't+b'). \tag{7}$$

Integrating equation (6) under the initial condition of $C_1 = 0$ at $t = 0$,

$$C_1 = \left\{C_s - b\left(\frac{V_0}{K_L A}\right) + a\left(\frac{V_0}{K_L A}\right)^2\right\}\left\{1-\exp\left(-\frac{K_L A}{V_0}t\right)\right\} - a\left(\frac{V_0}{K_L A}t\right) \tag{8}$$

VOL. 5 1961

Integrating equation (7) under the boundary condition of $C_2 = C_1$ at $t = t_1$,

$$C_2 = u\exp\left(-\frac{K_LA}{V_0}t\right) - a'\left(\frac{V_0}{K_LA}\right)t + \left\{C_s - b'\left(\frac{V_0}{K_LA}\right) + a'\left(\frac{V_0}{K_LA}\right)^2\right\}, \tag{9}$$

$$u = \left[(b'-b)\left(\frac{V_0}{K_LA}\right) + (a-a')\left(\frac{V_0}{K_LA}\right)^2 + (a'-a)\left(\frac{V_0}{K_LA}\right)t_1\right]\exp\left(\frac{K_LA}{V_0}t_1\right) +$$

$$+ b\left(\frac{V_0}{K_LA}\right) - a\left(\frac{V_0}{K_LA}\right)^2 - C_s.$$

As for the example shown in Fig. 4(a),

$a = -0{\cdot}2286$, $\quad a' = -0{\cdot}0006$,

$b = 1{\cdot}575$, $\quad b' = 0{\cdot}294$.

Substituting the above empirical data into equations (8) and (9) and taking $C_s = 6{\cdot}362$ ppm, the dissolved oxygen concentration curves were calculated as shown in Fig. 4(b), the parameter being K_LA/V_0. The data points of the dissolved oxygen concentration are also plotted in the same figure. Therefore, the overall coefficient of oxygen absorption of this example was estimated to be in a range from $^1/_4$ to $^2/_9$ per min.

The overall coefficients pertaining to the other experimental conditions were estimated following the same procedure. The results of these estimations are shown in Fig. 5, which shows a plot of the data in terms of K_LA/V_0 and V_s/V_0.

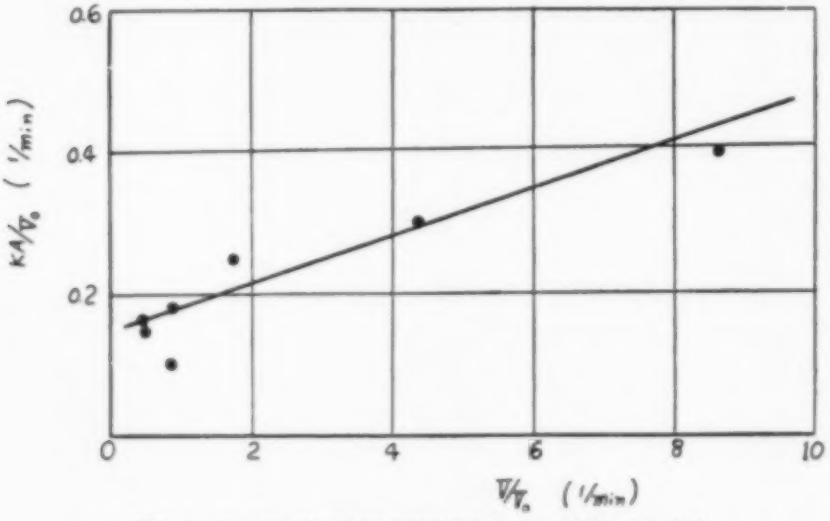


FIG. 5. An example of the relation between KA/V_0 and V/V_0.

The overall coefficients of oxygen absorption, thus estimated, are not free from end effects as discussed above. In this case, it does not seem to be significant to discuss the above mentioned effects separately, as done in equation (1).

The fact that a quantitative evaluation of an aeration equipment in practice following this procedure is considered to be of more importance in this section. The overall coefficient of oxygen absorption will depend on the geometry and the dimension of each specific aeration device and on the characteristics of bubbles, in particular (3).

An empirical relation $K_L A_0 \sim V_s^{0.5}$ is seen from Fig. 5. In this experiment a rather large vessel was used. Then, if an assumption that $K_L A_0 \gg K_L' A_0'$ could be made, the following correlation is obtained,

$$K_L A_0 \sim V_s^{0.5}. \tag{10}$$

Since K_L is independent of gas flow rates as described before,

$$A_0 \sim V_s^{0.5}. \tag{11}$$

The published paper of Hixson *et al.* (7) shows:

$$A_0 \sim V_s^{0.67 \text{ to } 0.82}. \tag{12}$$

The difference of exponent of V_s might be partly attributable to the fact that the mechanical agitation, together with aeration, was employed in the latter example. At any rate, Fig. 5 is concerned with a specific aeration device.

Using subscripts E and P to refer to experimental apparatus and the large scale device, respectively, a criterion for the scaling-up of this aeration device is assumed as follows:

$$(K_L A/V_0)_E = (K_L A/V_0)_P. \tag{13}$$

From equation (13).

$$\frac{(K_L A)_E}{(K_L A)_P} = \frac{(V_0)_E}{(V_0)_P},$$

$$\frac{(K_L A_0)_E + (K_L' A_0')_E}{(K_L A_0)_P + (K_L' A_0')_P} = \frac{(V_0)_E}{(V_0)_P}. \tag{14}$$

Assuming that $(K_L A_0)_P \gg (K_L' A_0')_P$,

$$\frac{(K_L A_0)_E + (K_L' A_0')_E}{(K_L A_0)_P} = \frac{(V_0)_E}{(V_0)_P}. \tag{15}$$

Considering that $(K_L)_P = (K_L)_E$ and rearranging equation (15),

$$\frac{(A_0)_P}{(A_0)_E} = \frac{(V_0)_P}{(V_0)_E[1 + (K_L' A_0')_E/(K_L A_0)_E]} = \frac{(V_0)_P}{(V_0)_E(1+\alpha)}, \tag{16}$$

where

$$\alpha = (K_L' A_0')_E/(K_L A_0)_E.$$

On the other hand,

$$A_0 = \pi d^2 n(H/v_b) = (V_s/10)\,(H/dv_b), \tag{17}$$

$$\bar{d} g \Delta\rho / v_b^2 \rho = 2, \tag{18}$$

$$\bar{d} = \beta V_s^{\gamma}, \tag{19}$$

VOL. 5 1961

where β = proportionality constant.

Substituting equations (17), (18) and (19) into equation (16) and considering a geometrical similarity between the experimental apparatus and a large-scale plant,

$$\xi = \frac{(V_g/V_0)_P}{(V_g/V_0)_E} = \left[(D_P/D_E)\ \exp\left(\frac{9\gamma-2}{2-3\gamma}\right)\right]\ldots[(1+\alpha)\ \exp\ (2)/(2-3\gamma)], \qquad (20)$$

where D = representative length of these devices. If the empirical data of γ and α are available, equation (20) is considered to indicate a basis to estimate the gas flow rate in the scaling-up.

NOMENCLATURE

a, b, a', b': empirical constants.
A_0: interfacial area between gas and liquid (cm²).
A'_0: equivalent interfacial area (cm²).
C, C_1, C_2: oxygen concentration in liquid (ppm).
C_s: equilibrium concentration of oxygen at an interface (ppm).
g: acceleration due to gravity (cm/sec²).
V_g: gas flow rate (cm³/min, l./min).
H: liquid depth (cm).
K_LA: $k_LA_0+k_L'A_0'$.
k_L: mass transfer coefficient of liquid film (cm/sec).
k_L' equivalent mass transfer coefficient (cm/sec).
n: number of bubbles per sec.
t: aeration time (sec, min).
V_0: liquid volume (cm³, l.).
v: nominal velocity of gas (cm/sec).
v_b: ascending velocity of bubble (cm/sec).
d: bubble diameter (cm).
$\bar{d}$: mean diameter of bubbles (volume–surface mean) (cm).
ρ: density of liquid (g/cm³).
$\Delta\rho$: density difference between liquid and gas.
μ: viscosity of liquid (c.p.).

REFERENCES

1. AIBA, S. *et al.*, Univ. of Tokyo, Biochem. Eng. Lab. Progress Report No. 5, No. 6 (1959).
2. CARVER, C. E., *Biological Treatment of Sewage and Industrial Wastes* p. 149. Reinhold, New York (1956).
3. ECKENFELDER, W. W., *J. Sant. Engng. Div., Proc. A.S.C.E.* **85**, No. SA4, (1959).
4. ECKENFELDER, W. W., *Sewage Industr. Wastes* **28**, 983 (1956).
5. IPPEN, A. T. *et al.*, *Sewage Industr. Wastes* **26**, 813 (1954).
6. VAN KREVELEN, D. W. *et al.*, *Chem. Engng. Progr.* **46**, 29 (1950).
7. HIXSON *et al.*, *Industr. Engng. Chem.* **42**, 1792 (1950).

EFFECT OF SURFACE ACTIVE AGENTS ON THE RATE OF OXYGEN TRANSFER

K. H. MANCY and D. A. OKUN
University of North Carolina

THE effect of anionic surfactants (Aerosol O.T. and alkyl benzene sulfonate, ABS) on the rate of oxygen transfer has been studied in bubble aeration processes. The influence of the surfactant concentration on the transfer rate as well as on the gas-liquid interfacial tension has been investigated. A discussion of the observed phenomena in terms of molecular structure of the interface, kinetics of oxygen transfer and its behavior in presence of surfactants is presented. A complete exposition of this study can be found in a report prepared by the authors (1).

The experiments were performed with an aeration column of 1·5 l. capacity and 50 cm height. Oxygen was introduced through a capillary or a diffuser at the bottom of the column. The oxygen concentration in the liquid was determined continuously by means of a specially designed polarographic cell. A recorder attachment allowed direct automatic recording of the rate of oxygen transfer. The apparatus used has been described in detail by the authors elsewhere (2).

VOL.
5
1961

Experiments in which oxygen was bubbled from a single capillary showed that on the addition of small amounts of Aerosol O.T. and ABS, there was a rapid decrease in the absolute rate of oxygen transfer (K_L)* to a minimum value. The minimum occurred at a concentration corresponding to the critical micelle concentration† (C.M.C.) of the surfactant. Further addition of the surfactant did not affect the absolute transfer coefficient (K_L).

On the other hand, increasing the surfactant concentration reduced considerably the gas–liquid interfacial tension which resulted in the formation of smaller bubbles for the same gas flow rates. This increase in the exposed interfacial area was accompanied by a gradual increase in the overall rate of oxygen transfer (K_La) at surfactant concentrations above the C.M.C. (3).

Experiments with a diffuser have shown that the K_La increases steadily with increasing surfactant concentrations despite the reduction in the K_L value. This is due to the effect of smaller bubbles produced by the diffuser and the great increase in the interfacial area with increasing surfactant concentrations.

* $KL = \frac{V}{A} K_La$ (cm/min),

where V = the volume of the liquid (cm³),
A = area of the gas–liquid interface (cm²),
K_La = the overall rate of oxygen transfer (min⁻¹).

† The critical micelle concentration (C.M.C) of a surfactant is the concentration at which the solution starts forming colloidal aggregates by the association of ions.

REFERENCES

1. OKUN D. A., MANCY K. H. and MCKEOWN J. J., *Oxygen in Waste Treatment, Effect of Surface Active Agents on Oxygenation.* Department of Sanitary Engineering, School of Public Health, University of North Carolina, Chapel Hill (Feb. 1959).
2. MANCY K. H. and OKUN, D. A., Automatic recording of dissolved oxygen in aqueous systems containing surface active agents. *Analyt. Chem.* **32**, 108 (1960).
3. MANCY K. H. and OKUN D. A., Effects of surface active agents on bubble aeration, *J.W.P.C.F.* **32**, 351 (1960).

EFFECTS OF SURFACE ACTIVE AGENTS ON OXYGEN BUBBLE CHARACTERISTICS

J. J. McKeown and D. A. Okun
University of North Carolina

Surface active agents have been observed to interfere with the rate of absorption of oxygen into aqueous solutions (1, 2). An attempt has been made to isolate those agents causing the greatest reduction in the rate of mass transfer (3). McKinney (4) has presented a tabulation of the volume of syndets produced, of which 80 per cent have anionic properties and 48 per cent of these anionic surfactants are alkyl benzene sulfonates, ABS.

BUBBLE CHARACTERISTICS

Several forces are exerted on a discrete bubble rising in an aeration column. Surface tension tends to keep the bubble in its most stable form, a sphere. The smaller the diameter of the bubble the greater is the effect of surface tension. Bouyancy, which is dependent upon bubble volume, causes the bubble to rise in the column, stabilizing at its terminal velocity a fraction of a second after its release from the forming device. The bubble loses oxygen through mass transfer during its rise, thus decreasing the mass that can exert pressure on the gas-liquid interface according to

$$PV \propto Nmc^2 \qquad (1)$$

where P = pressure (atm),

V = volume (ml),

N = number of molecules of the gas that travel in each principal direction, on the statistical average,

m = mass of one gas molecule,

c = root mean square speed of the molecules.

The loss of oxygen molecules is somewhat compensated for by the absorption of dissolved nitrogen and other gases from the solution into the oxygen bubble.

Another variable force exerted upon the bubble is the frictional resistance to rise of the gas through the liquid which is a function of the gas and liquid properties, cross-sectional area perpendicular to bubble flow and bubble volume. With larger bubbles, the resistance causes deformation of the bubble and a departure from a rectilinear rise to a helical rise.

The greater the bubble velocity, the more pronounced is the slipstream effect in increasing the velocity of the following bubble. Thus, bubbles have been observed to coalesce as they neared the top of a vertical aeration column. The slipstream effect is dependent on time of bubble formation, bubble size, and surface tension.

VOL. 5 1961

In dealing with pure solvents, an attempt has been made to relate orifice diameter and gas flow to the size of bubble produced (5–8). Fig. 1 shows the apparent relationships that exist between bubble size, frequency of formation and gas flow (9–12).

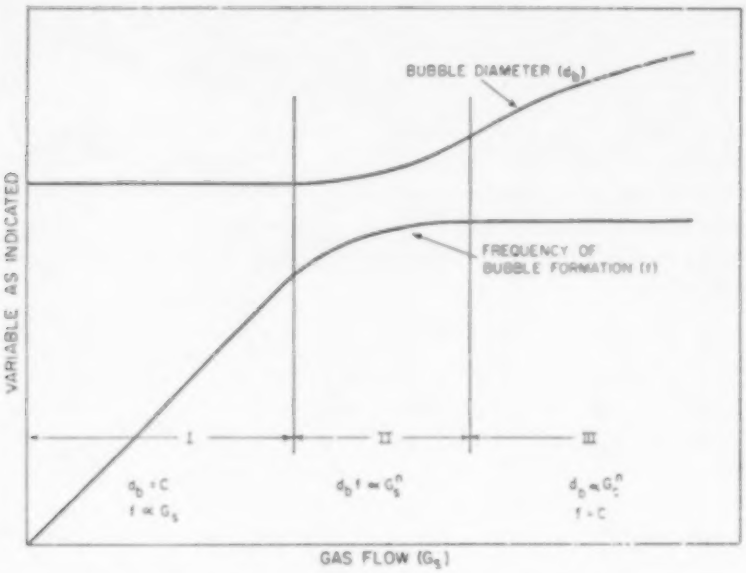


Fig. 1. The effect of gas flow on bubble characteristics.

Bubble velocity is primarily a function of bubble volume and frictional resistance. It has been stated that the so-called terminal velocity of rise occurs at a fraction of a millimeter of column height when the initial velocity of the bubble is in the range of 10–30 m/sec. This short deceleration is a function of the bubble size and independent of the initial velocity (13). Bubbles of 0·3 mm diameter have been observed as rigid spheres.

The fact that bubble velocity in distilled and tap water passes through a maximum with bubbles in the order of from 0·2 to 2 mm in diameter has been observed (14). Epstein (15) assuming the ratios of the viscosity and density of the gas to the liquid as small, approximates the velocity of a bubble to be

$$v = (g/3)\ (\rho'/\mu')r^2, \tag{2}$$

where v = terminal velocity of rise (cm/sec),

g = acceleration due to gravity (cm/sec^2),

ρ' = liquid density (g/ml),

μ' = liquid viscosity (g/cm sec) or (P),

r = bubble radius (cm).

Roesler (16), commenting on the above equation, concluded that under laminar flow conditions only, the bubble velocity was proportional to the square of the bubble radius.

Photographic studies of bubble behavior indicate that bubble shapes of various volumes in dynamic systems are anything but spheres. Most were elliptical or other symmetrical shapes (17). The kinetic energy of the bubble system aids in deforming bubble shapes (5). The actual bubble produced in such a system increases the surface area by about 10 per cent over and above that of the equivalent volume spheres (18). Bubble shape over a wide Reynolds number range showed little dependence on capillary and hydrodynamic conditions (19). Upon greatly increasing the liquid viscosity the bubble volume also increases. Bubble volume has been noted to be inversely proportional to the liquid density (5, 8).

THE EFFECT OF SURFACE ACTIVE AGENTS ON BUBBLE CHARACTERISTICS

Surface active agents exert considerable influence on the colligative properties of an oxygen–water interface. Surface active materials tend to form more plastic (less elastic) surfaces with increasing concentration. The surface viscosity acquires a definite yield value expressed as a function of the resistance to angular deformation (20, 21).

Many investigators have reported that lowering the surface tension of a solution by the introduction of various substances that adsorb at the gas-liquid interface is responsible for many effects on bubble characteristics (5–7, 10, 22–25).

Experimental apparatus

The apparatus used was essentially that described by Mancy and Okun (33). The determination of bubble velocity, shape, and size was accomplished by photographic techniques. A Kodak Signet 30 camera with a Portra portrait lens $8\frac{7}{8}$ in. from the subject was used. Two No. 1 photoflood lamps, each 36 in. from the subject served as the light source. A shutter speed of 1/250 sec allowed the observation of discrete bubbles.

Initially, metric rules were placed inside and outside of the column containing solution. The inside measurement was 1·0494 times larger than the external measurement. The scale inside the column was then removed to prevent disturbance to bubble flow.

The negative slides were projected and measurements made. The bubble, which resembled an oblate spheroid, is assumed to be circular in its horizontal projection. The bubble's major and minor axes were measured and the surface area of the bubble approximated by

$$a = m[(m^2 + f^2)/2]^{1/2} \tag{3}$$

where f and m are the minor and major axes, respectively.

Results and discussion

Fig. 2 indicates that there was no appreciable change in bubble volume with increasing gas flow for all ABS concentrations studied. On the other hand, frequency of bubble release was directly proportional to gas flow. The lowest gas flows studied

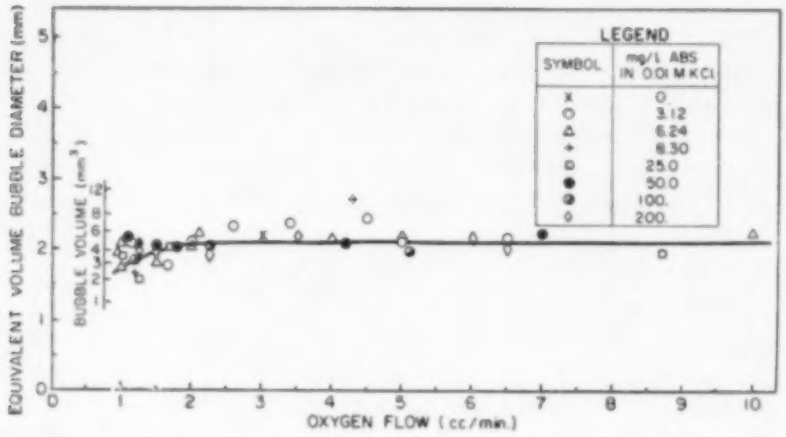


FIG. 2. The effect of ABS concentration on bubble size at low gas flow.

showed a minor dependence of bubble diameter on surface active agent concentration. This is possibly the result of the slow rate of adsorbtion of ABS. Only when the time of bubble formation was very low was the ABS instrumental in decreasing bubble size.

In considering the complete gas flow range as including three zones, the first zone represents the greatest surface area produced per unit gas flow. As the gas flow increases through zones II and III the surface area produced per unit gas flow decreases (Fig. 1).

Bubble shape is a prominent factor in the relationship between diameter and velocity. Small bubbles tend to be spherical. Upon increasing the diameter the bubble becomes ellipsoidal and at large size it assumes a capped shape. In each shape zone the bubble has a distinctive motion. The larger the bubble the more helical the pattern of rise. Therefore the bubble seems to inflict upon itself a condition of turbulence and although the time taken for the larger bubble to travel a fixed vertical distance is recorded as the velocity, the bubble actually travels a much longer path, considering the third dimension. It appears that the velocity zones would be less pronounced if the deviation from rectilinear rise were taken into account. The fact that the bubble contacts more bulk liquid because of its rocking or helical motion increases its oxygen transfer efficiency.

A study was conducted to show variation in bubble shape as a function of hydrostatic head. Pictures were taken of various bubbles as they traveled the height of the aeration column. Actual surface area and volume were calculated. The bubble was observed to approach spherical dimensions as it rose. The bubbles had approxi-

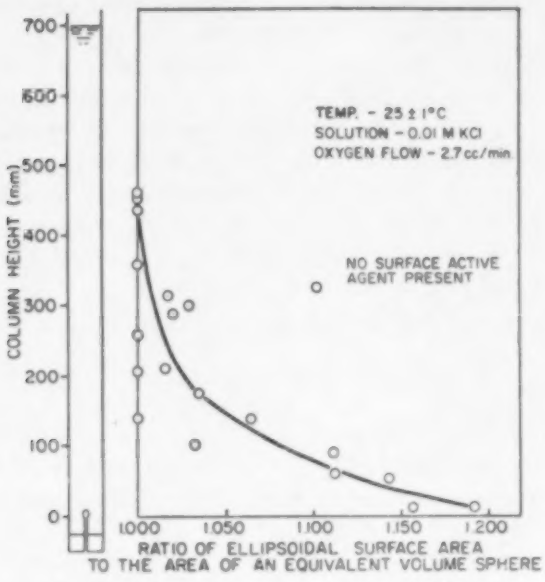


FIG. 3. Variation in shape of a rising bubble.

VOL. 5 1961

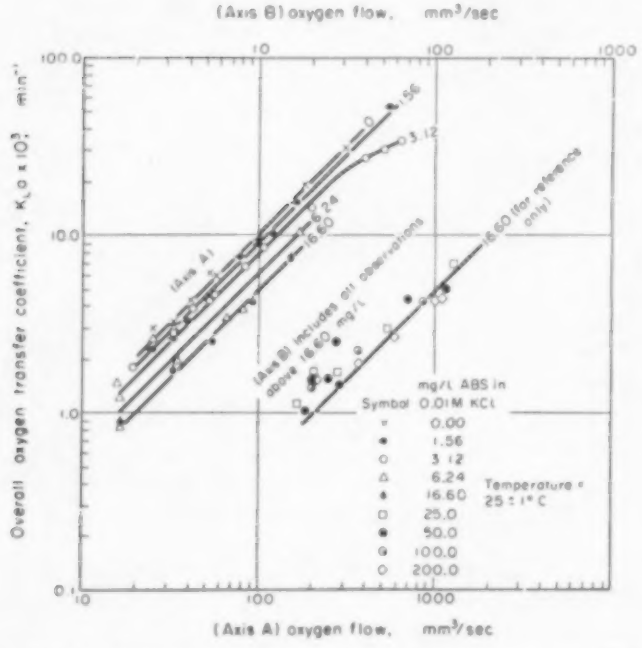


FIG. 4. The effect of ABS concentration and oxygen flow on the overall oxygen transfer coefficient.

mately 20 per cent more surface area on formation than they had after they traveled two-thirds of the column height. The scattering of points may be due to bubble oscillation after formation. During the first oscillation the amplitude and wavelength are maximum. After the first wave is completed both amplitude of shape and distance traveled decrease with time until the bubble stabilizes at its equilibrium shape. Fig. 3 shows shape varying in 0·01 M KCl, where the final shape is that of a sphere due to high surface tension. The final shape of bubbles rising in a solution of surface active agent is dependent on the bubbles' equilibrium surface tension.

Fig. 4 shows the dependence of the overall oxygen coefficient, K_La, on gas flow for ABS concentrations of from 0 to 16·6 mg per l. The decrease in K_La with increasing surface active agent concentration is typical of previous results reported. The relationship may be expressed as

$$K_La = CG_s, \tag{4}$$

where the gas flow exponent is unity.

Additional quantities of ABS from 25 to 200 mg per l. produces a slight increase in K_La at the lower gas flows. Fig. 4 shows these data plotted about the reference line corresponding to 16·6 mg per l. ABS.

It is well to point out that bubble splitting at the higher concentrations was kept to a minimum due to near laminar conditions of flow. Therefore, the recovery of the K_La was kept to a minimum, as bubble splitting is primarily a function of the combined low surface tension and liquid turbulence.

The ratio of bubble width to equivalent volume diameter is a measure of the deviation from spherical shape. Figure 5 shows this ratio as a function of ABS

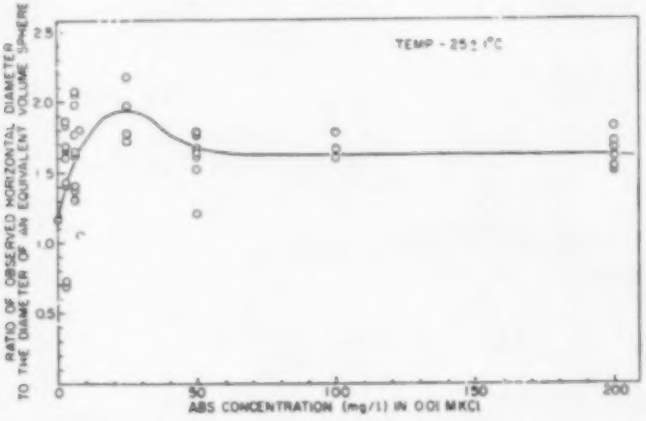


Fig. 5. Effect of ABS concentration on bubble shape.

concentration. The bubble becomes less spherical as surface active agent concentration increases. This effect is directly related to the loss of surface energy as shown by the decrease in surface tension.

This result, however, is opposite to the observations of Baars (25) who showed

VOL.
5
1961

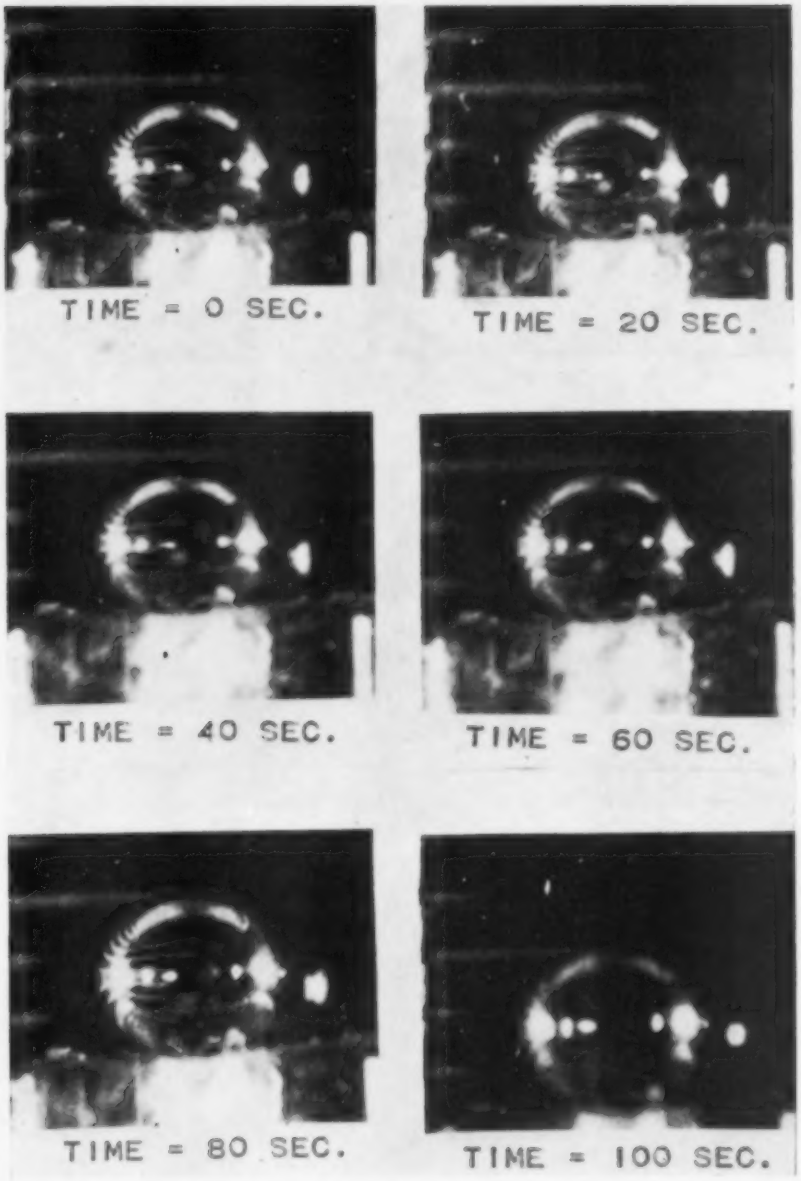


FIG. 6. Increase in volume after formation of an air bubble in 6·24 mg/l. ABS.

a nitrogen bubble to approach spherical dimensions upon the addition of surface active agent.

Viscosity determinations were made at all ABS concentrations and showed no appreciable change.

SURFACE AGING OF A BUBBLE

The basic principles which account for the reduction of the oxygen transfer rate may be described by considering interfacial forces. The gas–liquid surface in water possesses a given energy in the plane of the surface. The surface energy is a function of the attractive forces existing within the surface which, in turn, are a function of the summation of the dipolar moments affecting the surface.

When a surface active substance is added to the solution, the molecules migrate and orient themselves to the surface in accordance with their polar properties. The rate at which the molecules adsorb on the surface is of particular importance in an aeration system. A number of surface active agents adsorb rapidly while others vary from fast to slow.

A 3-mm diameter oxygen bubble will have a velocity in water of approximately 0·8 ft per sec. Therefore, in a 15-ft deep aeration tank the bubble's life after formation will be about 19 sec. Those surface active agents that take longer than this time to adsorb under turbulent conditions will have little effect on the rate of oxygen transfer.

VOL. 5 1961

It has been noted that the maximum transfer from bubbles occurs during and immediately after formation. Should the surface active agents in solution not have time to collect at the gas–liquid interface due to their slow adsorption rate, oxygen transfer will not be depressed during the critical phase of bubble formation.

The advantage of a highly turbulent aeration system such as brush or turbine aeration where surface renewal is rapid is of great importance. Several blades spaced vertically on the same shaft will increase the area of new surface formed by destroying old surfaces at various tank depths.

It is rational to assume that the amount of material present at a gas–liquid interface is a function of the surface active substance, the instantaneous surface tension, and the bulk concentration. The effect of temperature and rate of change of surface tension for some surface active agents has been reported (26, 27). The slow rate of adsorption of some surface agents is thought to be a result of slow reorientation at the surface in addition to the dehydration of the hydrophobic portion of the molecule (27, 28).

Apparatus has been described for dynamic surface tension measurement for surfaces at various ages (29, 30).

Force area curves showed that the surface area per molecule increased with increasing molecular weight and would explain the delay in formation of micelles at low molecular weights (31).

A summary of surface energies is given by Weiser (32). It is shown that low concentrations of highly surface active agents lower the surface tension of water markedly. Rising bubbles carried from two to eight times as much solute at the

interface as would be predicted by Gibb's equation. The suggestion is made that under dynamic conditions the chains of oriented molecules extend downward into the solution from the outermost monomolecular fiber. Static surfaces have been observed to adhere to Gibb's law more nearly than dynamic surfaces. Dynamic surface tensions seem to be nearer that of the solvent than the static measurements.

Experimental procedure

The purpose of this experiment was to determine the change in surface tension with time for various concentrations of ABS. A single air bubble was produced that adhered to the forming orifice. Gas flow to the bubble was stopped and the bubble was observed under a fixed hydrostatic head. Pictures were taken every 5 sec until the bubble volume became large enough to escape from the forming orifice.

The photographs were enlarged ten times their actual size and the vertical projection of the bubble was planimetered. The solution was saturated with oxygen from air to eliminate oxygen transfer.

Results

Fig. 6 shows a sample set of bubbles in 6·24 mg per l. ABS increasing in volume with time. Fig. 7 illustrates the area increase and shape variation as observed. This single experiment is preliminary but shows the possibilities of measurement of

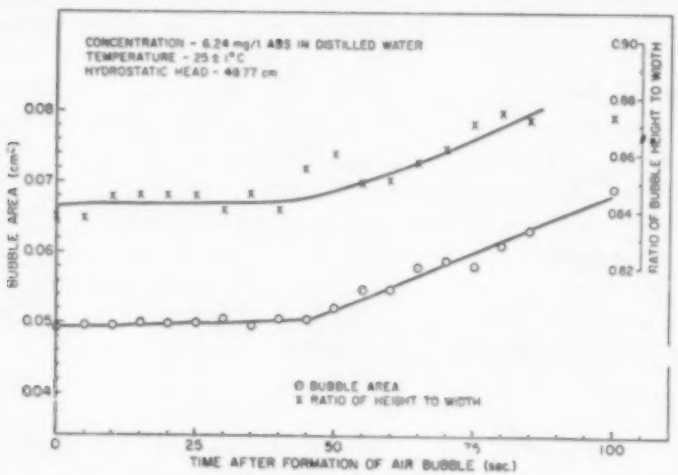


Fig. 7. Growth of a single air bubble with time due to decreasing surface tension.

dynamic surface tension and rate of adsorption in bubble aeration. The procedure may be improved to stabilize initial bubble volume.

Under the test conditions the resultant of the bouyant bubble force must be equal to the summation of all forces operating in the opposite directions at release, namely hydrostatic head and surface tension.

SUMMARY AND CONCLUSIONS

The following results have evolved from a photographic study of bubble characteristics:

At low gas rates, bubble size is constant while frequency of bubble production is directly proportional to gas flow rate. At high gas flow rates, bubble size increases with increasing gas flow.

The bubbles observed in 0·01 M KCl and various ABS concentrations were pulsating ellipsoids. The greatest surface area per bubble was shown to occur immediately after formation. The addition of ABS to the 0·01 M base solution resulted in producing a more elliptical bubble.

The bubble became more plastic upon the addition of ABS and bubble splitting was observed at the higher surfactant concentrations.

The overall oxygen transfer coefficient, $K_L a$, varied directly with gas flow rate for all ABS concentrations.

The ratio of observed bubble width to equivalent volume diameter increased to a maximum at the critical micelle concentration for ABS and decreased slightly at higher concentrations.

An experimental procedure has been presented for evaluation of dynamic surface tension and rate of adsorption of surface active molecules. The procedure is believed to produce more reliable surface tension measurements than the ring method when applied to bubble aeration.

Acknowledgement—This work is supported in part by a grant RG-3720 from the National Institutes of Health of the Public Health Service.

VOL. 5 1961

REFERENCES

1. Camp, T. R., Gas transfer to and from aqueous solutions. *Proc. Amer. Soc. Civ. Engrs.* **84**, No. SA4, Paper 1701 (1958).
2. Downing, A. L. and Truesdale, G. A., Some factors affecting the rate of solution of oxygen in water. *J. Appl. Chem.* **5**, 570–81 (1955).
3. Sawyer, C. N., Effects of synthetic detergents on sewage treatment processes. *Sewage and Industr. Wastes* **30**, 757–75 (1958).
4. McKinney, R. E., Syndets and waste disposal. *Sewage Industr. Wastes* **29**, 654–66 (1957).
5. Jackson, R. W., The formation of gas bubbles from orifices. *Industr. Chem.* **28**, 346–51 (1952).
6. Benzing, R. J. and Myers, J. E., Low frequency bubble formation at horizontal circular orifices. *Industr. Engng. Chem.* **47**, 2087–90 (1955).
7. Haney, P. D., Theoretical principles of aeration. *J. Amer. Wat. Wks Ass.* **46**, 353–76, (1954).
8. Coppock, P. D. and Meiklejoh, G. T., Behavior of gas bubbles in relation to mass transfer. *Trans. Inst. Chem. Engrs.*, (*London*) **29**, 75–86 (1951).
9. Verschoor, H., Some aspects of motion of a swarm of gas bubbles rising through a vertical liquid column. *Inst. Chem. Engr.*, (*London*) **14**, 42–6 (1950).
10. Smirnov, N. I. and Ruban, V. L., Movement of bodies in a medium. *J. Appl. Chem.*, (*USSR*) **26**, 95–8 (1953).
11. Eckenfelder, W. W., Absorption of oxygen from air bubbles in water. *J. Sanit Engng. Div. ASCE* **85**, 89–99 (1959).
12. McKeown, J. J. and Okun, D. A., Absorption of oxygen from air bubbles in water. *J. Sanit. Engng. Div. ASCE* **86**, 37–42 (1960).

13. Vitkina, M. A., Motion of gas bubbles in the bubbling process. *Zh. Prikl. Khim.* **24**, 1041–4 (1951); *Chem. Abstr.* 47–5212d.
14. Carver, C. E., Absorption of oxygen in bubble aeration. *Biological Treatment of Sewage and Industrial Wastes* vol. 1. Reinhold, New York (1956).
15. Epstein, P. S. and Plesset, M. S., The stability of gas bubbles in liquid gas solutions. *J. Chem. Phys.* **18**, 1505 (1950).
16. Roesler, F. C., Gas bubbles in solution. *J. Chem. Phys.* **19**, 512–13 (1951).
17. Helsby, F. W. and Tuson, K. R., Behavior of air bubbles in aqueous solutions. *Research* **8**, 270–5 (1955).
18. Quigley, C. J., Johnson, A. I. and Harris B. L., Size and mass transfer studies of gas bubbles. *Chem. Engng. Symp. Series* **16**, 51 (1951).
19. Ladyzhenskii, R. M., Investigation of the shape of a gas bubble rising in a stationary fluid. *Zh. Prikl. Khim.* **29**, 193–200 (1956); *Chem. Abstr.* 50–13557b.
20. Burcik, E. J. and Newman, R. C., The surface plasticity of a homologous series of sodium soap solutions. *J. Colloid Sci.* **12**, 10–18 (1957).
21. Nielsen, L. E., Wall, R. and Adams, G., Coalescence of liquid drops at oil-water interfaces. *J. Colloid Sci.* **13**, 441–58 (1958).
22. Garner, F. H. and Hammerton, D., Circulation inside gas bubbles. *Chem. Engng. Sci.* **3** 1–11 (1954).
23. Bickerman, J. J., *Surface Chemistry* (2nd ed.). Academic Press, New York (1958).
24. Pattle, R. E., Aeration of liquids. *Trans. Industr. Chem. Engrs. (London)* 17–27 (1950).
25. Baars, J. K., The effect of detergents on aeration: A photographic approach to the problem. *Proc. Inst. Sewage Purif.* pt. 4, 358–62 (1955).
26. Crockford, H. D. and Knight, S. B., *Fundamentals of Physical Chemistry*. John Wiley, New York (1959).
27. Alexander, A. E., The surface ageing of solutions. *Trans. Faraday Soc.* **37**, 15–25 (1941).
28. Brady, A. P. and Brown, A. G., Mechanical properties of the surface films on aqueous solutions of detergents. *Monomolecular Layers*. American Association for the Advancement of Science (1954).
29. Addison, C. C. and Elliott, T. A., The properties of freshly formed surfaces—X. *J. Chem. Soc.* **41**, 2789–2805 (1949).
30. McBain, J. W. and Bolduan, O., Osmotic properties of solutions of some typical colloidal electrolytes. *J. Phys. Chem.* **47**, 94–103 (1943).
31. Meader, A. L. and Criddle, D. W., Force-area curves of surface films of soluble surface active agents. *J. Colloid Sci.* **8**, 170–8 (1953).
32. Weiser, H. B., *A Textbook of Colloid Chemistry*, John Wiley, New York (1958).
33. Okun, D. A., Mancy, K. H. and McKeown, J. J., *Oxygen in Waste Treatment, Effect of Surface active Agents on Oxygenation*. Department of Sanitary Engineering, School of Public Health, University of North Carolina, Chapel Hill (1959).

EFFECTS OF SURFACE ACTIVE AGENTS ON RE-AERATION

DONALD J. O'CONNOR

Manhattan College

REVIEW OF PREVIOUS WORK

UNPOLLUTED water maintains in solution the maximum quantity of dissolved oxygen, which is in equilibrium with the partial pressure of oxygen in the atmosphere. When dissolved oxygen is removed from the solution, an unbalance is created and the deficiency is made up by the atmospheric oxygen passing into solution. The rate at which re-aeration takes place is proportional to the dissolved oxygen deficit and may be expressed as:

$$\frac{dD}{dt} = -K(C_s - C) = -KD, \tag{1}$$

in which $\frac{dD}{dt}$ = rate of re-aeration,
C_s = dissolved oxygen saturation,
C = dissolved oxygen concentration,
D = dissolved oxygen deficit,
K = coefficient of re-aeration, base e.

Equation (1) is based on the assumption that the concentration of dissolved oxygen is uniform throughout the depth of the water body. K, the re-aeration coefficient, is referred to as the volumetric coefficient. This coefficient may be expressed in terms of the transfer coefficient, K_L, as follows:

$$K = K_L \cdot \frac{A}{V}, \tag{2}$$

in which A = surface area,
V = volume of liquid,
K_L = oxygen transfer coefficient.

Lewis and Whitman (1) presented a model to define the transfer coefficient. This theory is based on the assumption that there are two films located at the air–water interface, through which the gas must pass by molecular diffusion. For gases of low solubility, such as oxygen and carbon dioxide, the liquid film is the controlling resistance and the transfer coefficient may be written:

$$K_L = \frac{D_L}{Y}, \tag{3}$$

VOL. 5 1961

in which D_L = coefficient of molecular diffusion of oxygen,
Y = film thickness.

The most significant development in recent years has been presented by Danckwerts (2) who proposed a theory defining the transfer coefficient as follows:

$$K_L = [D_L r]^{\frac{1}{2}}, \tag{4}$$

in which r = rate of surface renewal.

The effect of various substances such as organics and surface active agents on the transfer or re-aeration coefficient has been recognized and reported (3, 4). In general these substances alter either the viscosity or the surface tension of the solution. Any change in the viscosity affects the diffusion coefficient of the gas in the liquid and consequently influences the transfer rate. It is assumed that this effect may be approximated by the following relationship:

$$D_L = A_1 c^{-b}, \tag{5}$$

in which A_1 = constant,
c = concentration,
b = exponent.

Substances which influence the surface tension of solutions also affect re-aeration rates, inasmuch as they produce changes in the surface characteristics of the solution. Surface active substances tend to concentrate at the interface and apparently create a barrier to the oxygen diffusion. The magnitude of this influence is a function of the type and concentration of the substance. The surface concentration of these substances is greater than that of an equal mass in the body of the solution. The excess surface concentration is related to the change in surface tension, which decrease with increasing concentration of surface active substances. This condition is referred to as positive absorption and is characteristic of solutions of organic substances, soaps and detergents. On the other hand, inorganic ionized substances are negatively absorbed. They are drawn into the bulk of the solution and the surface layer has a concentration less than that of the bulk. In this case, surface tension increases with increasing concentration. This phenomenon is described by the Gibbs adsorption equation as follows (5):

$$\Gamma = -\frac{c}{RT}\frac{d\gamma}{dc}, \tag{6}$$

in which Γ = excess concentration in the surface layer,
c = concentration in the bulk of solution,
γ = surface tension,
T = absolute temperature,
R = gas content.

Γ is positive when the differential of surface tension with respect to concentration is negative, i.e. when the surface tension decreases with concentration. Conversely Γ is negative, when there is an increase in the surface tension with concentration.

THEORETICAL DEVELOPMENT

Based on the assumption that the diffusion is molecular in nature through a very thin film at the liquid–air interface, it follows that substances which concentrate at the interface interfere with the transfer of gas from the atmosphere to the liquid.

It is assumed that such substances have a two-fold effect on gas transfer by virtue of their effect on: (1) the diffusion coefficient, which is reduced with increasing concentration because of the change in the surface or bulk viscosity; (2) the film thickness, which varies with the excess surface concentration. Since a defined surface film exists in solution of surface active substances, some form of equation (3) may be assumed as an appropriate model. In accordance with this model and the above assumptions, the reduction in the transfer coefficient varies inversely as the diffusion coefficient and directly as the excess surface concentration:

$$\Delta K \propto \frac{\Gamma}{D_L}. \tag{7}$$

Substituting for Γ its value from the Gibbs equation (equation 6) and for D_L its value as given by equation (5), and combining constants and exponents, the following equation is obtained:

$$\Delta K = -Ac^n \frac{d\gamma}{dc}, \tag{8}$$

in which $\Delta K = K_0 - K_c$ = change in transfer coefficient,
K_0 = coefficient at zero concentration,
K_c = coefficient at concentration c,
A = constant (for a given temperature),
n = exponent = $b+1$.

Depending on the relationship between the concentration and surface tension, $d\gamma/dc$ will have various forms. A relationship between these variables cannot as yet be expressed as a general equation valid for all solutes, although many empirical relationships (5) have been proposed.

A function which defines the surface tension data of the solutions dealt with In this work is:

$$\frac{d\gamma}{dc} = -f_m[\gamma - \gamma_\infty]\, m = -f_m \sigma m, \tag{9}$$

in which γ = surface tension,
γ_∞ = limiting surface tension,
σ = $\gamma - \gamma_\infty$
c = concentration,
f_m = constant,
m = exponent.

From a preliminary analysis of the solutions investigated in this work it appears that the exponent m has a value of between 1 and 2.

VOL. 5 1961

Substitution of equation (9) for $m = 1$ and $m = 2$ in equation (8), respectively, yields:

$$K_0 - K_c = \Delta K = A f_1 c^n [\gamma - \gamma_\infty], \tag{10a}$$

and

$$\Delta K = A f_2 c^n [\gamma - \gamma_\infty]^2. \tag{10b}$$

The final forms of the equations are obtained by substituting the integrated forms of equation (9) in equations (10a) and (10b) and rearranging terms:

$$K_c = K_0 - \frac{A f_1 \sigma_0 c^n}{{}_e f_1 c}, \tag{11a}$$

and

$$K_c = K_0 - \frac{A f_2 \sigma_0^2 c^n}{[f_2 c \sigma_0 + 1]^2}, \tag{11b}$$

in which $\sigma_0 = \gamma_0 - \gamma_c$,

γ_0 = surface tension of water.

The constants A and f are related to the nature and type of surface active substance. The constant f may be determined from published surface tension data or from laboratory measurements and the constant A from the experimental data.

EXPERIMENTAL APPARATUS AND PROCEDURE

The experimental procedure consisted in determining the re-aeration coefficient in pure water and in solutions of various concentrations of organic substances and detergents. An experimental apparatus was employed, in which all other factors which would influence the magnitude of the re-aeration coefficient were held constant, i.e. the rate of surface renewal and the ratio of the surface area to the volume.

The experimental apparatus consisted of a test cylinder which contained approximately 1600 ml of the various solutions. A uniform concentration of dissolved oxygen throughout the volume of solution was maintained at any instant by means of a propeller. The submergence of the liquid over the propeller at rest was 4 in. and approximately 1 in. during rotation. The concentration of dissolved oxygen was determined polarographically. Tests were conducted on solutions of sodium lauryl sulfate, peptone, caproic and heptanoic acid, all of which are surface active in nature. Inorganic salt solutions of potassium chloride and magnesium carbonate were also tested, as examples of negative adsorption. The coefficient was determined by means of the integrated form of equation (1). A test run was in the order of from 30 min to 1 hr, during which period approximately 60 per cent–80 per cent re-aeration took place. Samples were taken every 5 to 10 min. At least five values were read for each run. The polarograph was calibrated for each solution employed. Polarograms for each solution were determined and appropriate voltages selected. Values of surface tension were determined by means of a du Nouy tensiometer. Temperature variations were within 0·5°C±. A complete description of the laboratory procedure and apparatus is given elsewhere (6).

DISCUSSION OF RESULTS AND ANALYSIS OF DATA

For each substance tested, data on the re-aeration coefficients and surface tensions are available for different concentrations. With these data the analysis proceeds as follows:

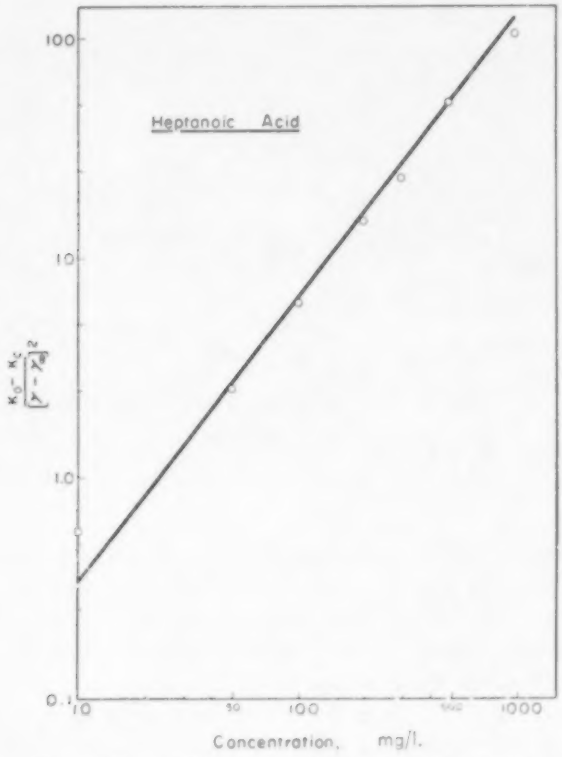


FIG. 1. Heptanoic acid—graphical correlation of data.

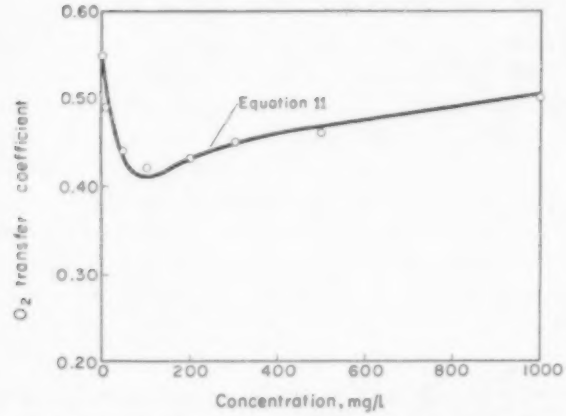


FIG. 2. Heptanoic acid—vs. concentration re-aeration coefficients.

VOL. 5 1961

(1) It is first necessary to assume a value of the limiting surface tension, if this is not available from the laboratory data. This value is subtracted from each of the measured surface tension values. The logarithm of this difference (for $m = 1$) or the reciprocal of this difference (for $m = 2$) is plotted against the associated concentration. The slope of the line is a measure of the constant, f_1 or f_2.

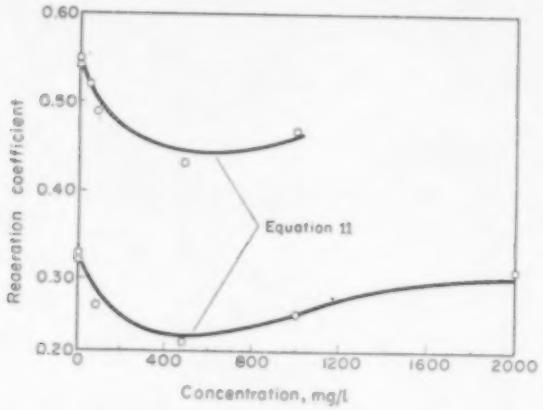


FIG. 3. Caproic acid—vs. concentration re-aeration coefficients.

(2) Based on the relationship, as determined in step 1, the surface tensions are calculated for each substance. The ratios of the ΔK and $f_1 (\gamma - \gamma_\infty)$ or $f_2 (\gamma - \gamma_\infty)^2$ are plotted against the concentrations on log–log paper. The power, n, of the concentration and the constant, A, are determined from this plot. An example of this correlation for heptanoic acid is shown in Fig. 1.

(3) Equations (11a) and (11b) may be directly employed to calculate the aeration coefficient for various concentrations of the surface active agent. Comparisons are made between the calculated results and observed data and an example of this comparison of the heptanoic acid data is shown in Fig. 2. In Fig. 3 are shown the re-aeration coefficients for various concentrations of caproic acid, calculated by the same procedure. The data for peptone which showed a constant value for the re-aeration coefficient at low concentrations up to 200 ppm, evidenced a slow decrease after this point. At a concentration of 2500 ppm, no rise in the curve was evident. The re-aeration data for the solutions of potassium chloride and magnesium carbonate indicate slight changes. The differences, however, were not sufficiently significant to draw any conclusions. Preliminary runs on other detergents were conducted, but the data were inconsistent and reproducibility was difficult to achieve in these cases. This condition was also evident on the runs of the organic acids and sodium lauryl sulfate.

Data from another source (4) were also available to test the proposed hypothesis. These experiments were conducted in order to determine the effects that surface active agents have on the carbon dioxide transfer coefficient. The data were analyzed

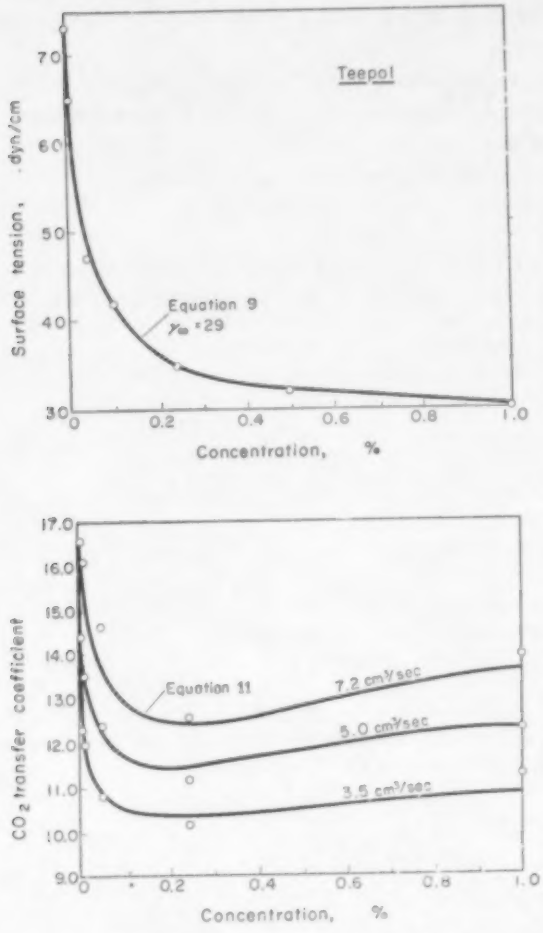


FIG. 4. Teepol—surface tensions and CO_2 transfer coefficients.

as described in steps 1, 2 and 3. The comparison between the calculated and observed values of the transfer coefficient for different flow conditions is shown in Figure 4 for various concentrations of the commercial detergent, Teepol.

CONCLUSIONS

From the results of this and other studies, the following conclusions may be drawn:

(1) The addition of surface active agents causes a reduction in the oxygen transfer coefficient in surface aeration to a minimum value. Further addition causes a rise in curve to a value which approaches that of pure water.

(2) This phenomenon may be related to the excess surface concentration as

VOL. 5 1961

defined by the Gibbs absorption equation and to the variation of the oxygen diffusion coefficient.

(3) The proposed hypothesis provides some insight into the nature of this interference. The equations, which are developed from the hypothesis, agree reasonably well with the experimental data.

(4) Further work should be conducted covering a wide range of substances to verify the hypothesis proposed in this work.

Acknowledgement—This work was supported by a research grant (Rg 4673) from the National Institutes of Health, U.S. Public Health Service. The assistance of the following is gratefully acknowledged: Edwin Barnhart, research assistant, who worked on the development of the experimental apparatus and the preliminary procedures and to William Keane and David Romano, students, who assisted in the laboratory experiments. The latter was supported in part by a student research participant grant from the National Science Foundation.

REFERENCES

1. LEVIS W. K. and WHITMAN W. G., Principles of gas absorption. *Industr. Engng. Chem.* **16**, 1215 (1924).
2. DANCKWERTS P. V., Significance of liquid film coefficients in gas absorption. *Industr. Engng. Chem.* **43**, No. 6 (1951).
3. HOLROYD A. and PARKER H. A., Investigations on the dynamics of aeration—The effects of some surface contaminants. *J. Inst. Sewage Purif.* 280 (1952).
4. CULLEN E. J. and DAVIDSON J. F., The effect of surface active agents on the rate of absorption of carbon dioxide by water. *Chem. Eng. Sci.* **6**, No. 2 (1956).
5. BIKERMAN J. J., *Surface Chemistry*. Academic Press, New York (1958).
6. O'CONNOR D. J., *Effect of Physical and Chemical Variables on the Re-aeration Coefficient*. Progress Report on Research Contract KI-CI. National Institute of Health, USPHS (1959).

OXYGEN TRANSFER IN THE ACTIVATED SLUDGE PROCESS

A. L. Downing and A. G. Boon

Water Pollution Research Laboratory, Stevenage, England

VOL. 5 1961

INTRODUCTION

Work has been in progress at the Water Pollution Research Laboratory in the last few years to evaluate the performance of aerators of all types and to establish the fundamental principles governing the rate of mass-transfer of oxygen between air and aqueous liquids, and from the liquid to micro-organisms within it, upon which this performance depends. The ultimate object is to provide information from which it is hoped it will be possible to reduce the costs of aeration in the activated-sludge process or to produce effluents of better quality for the same cost.

The work has several facets since the performance of an aeration unit cannot be defined entirely in terms of the rate and efficiency with which oxygen is absorbed from the air into the liquid, although these criteria are clearly of major importance; other factors such as the effects of agitation on the degree of longitudinal mixing, and on the reactivity or settleability of the sludge, could affect the quality of the effluent and must be taken into account in any comprehensive assessment.

In this present paper the results of the work on oxygen-transfer at the air–water interface are presented and their relation to information from other sources indicated where appropriate.

EXPERIMENTAL METHODS

The experimental methods used for measuring the performance of aerators have already been described elsewhere (1). Rates of absorption of oxygen were determined by well established procedures (2, 3) involving measurement either of the increase of the concentration of dissolved oxygen in the previously deoxygenated liquid during aeration for a given period (the "unsteady-state" method) or, when the liquid contained activated sludge, of the reduction in the oxygen content of the air in contact with the liquid and the equilibrium concentration of dissolved oxygen maintained in the liquid under steady-state conditions. In determining the performance of full-scale diffused-air units at sewage works by the latter method the issuing gas was collected under a light floating hood (Fig. 1) from which it was pumped to a paramagnetic analyser for determination of the depletion of oxygen (Fig. 2). In order to be able to calculate the rate of absorption of oxygen it was also necessary to know the flow of air. At some sewage works where the flow was not metered an approximate estimate was obtained by first determining the proportion

of carbon dioxide in the gas with a recording analyser, then injecting a known flow of carbon dioxide into the outlet pipe from the hood, and finally redetermining the proportion of carbon dioxide at a point downstream (Fig. 2).

Power requirements of mechanical aerators were determined by the use of dynamometers or by measurement of the consumption of electrical energy; those of diffused-air systems were calculated assuming "polytropic" compression (1, 4).

Rates of solution of oxygen have been expressed mainly in terms of oxygenation capacities (5) (in g/hr) and, in the case of diffused-air systems, in terms of a diffused-air coefficient, K_A, defined as the volume of oxygen absorbed per unit volume of air diffused per unit oxygen deficit per unit depth of liquid through which the bubbles rise; the units chosen for K_A are those of ml/100 ml ppm m. Mass-transfer coefficients (K_L, in cm/hr) have also been determined for some systems.

Aeration efficiencies (the oxygenation capacities attainable per unit input of power) have been given in terms of g/kWh (1000 g/kWh ≡ 1·64 lb/h.p. hr). To distinguish between the aeration efficiency when assessed in terms of the total energy consumption of the power unit and when expressed in terms of the net energy actually used in aeration, the words "practical" and "theoretical" are used.

In calculating the mass-transfer coefficient, K_L, or the overall absorption coefficient, K_La, from measurements of the rate of uptake of oxygen in diffused-air plants by the unsteady-state method, it is necessary to take into account that the composition of the gas stream will vary during the experiment. This has been done by combining the well known rate equation,

$$dC/dt = K_La(C_{SB}-C), \tag{1}$$

with a mass balance

$$V(dC/dt) = \rho Q(x_I-x_0) \tag{2}$$

and with the equilibrium relations

$$2C_{SB}/(x_I+x_O) = C_{SBU}/x_I = H. \tag{3}$$

Eliminating C_{SB} and integrating leads to

$$K_La = \frac{1+(K_LaVH/2\rho Q)}{t} \cdot \log_e \frac{C_{SBU}}{C_{SBU}-C} \tag{4}$$

with the aid of which K_L or K_La can be evaluated.

In the above equations C_{SB} is the saturation concentration in water in equilibrium with the gas at any time t, when the proportion of oxygen by volume in the issuing gas is x_O, C_{SBU} is the corresponding concentration when the liquid has reached equilibrium with the inflowing gas in which the proportion of oxygen is x_I, V is the volume of liquid, Q is the flow of air, ρ the density of gaseous oxygen at the temperature and pressure at which the air flow is expressed, and H is a constant. In deriving equations (2) and (3) it is assumed that the flow of air entering the unit is the same as that leaving and that the saturation concentration at any time is proportional to the arithmetic mean of the proportions of oxygen in the inflowing

VOL.
5
1961

FIG. 1. Measuring the performance of full-scale diffused-air activated-sludge plants.

and outflowing gas. Neither assumption is strictly correct but the error introduced will normally be negligible.

It has been the practice to compare the performance of different systems using a liquid of substantially constant composition, that chosen being tap water containing 5 ppm anionic detergent (expressed as Manoxol O.T. (6)). The main reason for this choice was that the rate of absorption of oxygen from diffused-air bubbles

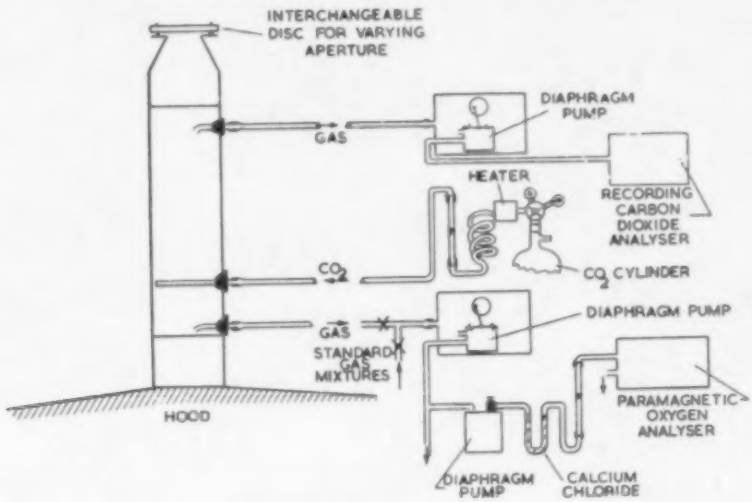


FIG. 2. Apparatus used for measuring rates of aeration in diffused-air units at sewage works.

in this liquid was similar to that in mixtures of sewage and activated sludge containing 5 ppm anionic detergent, this concentration being about the average in sewage works at the time the investigation was started. However, it was evident that this average concentration would be unlikely to remain constant and supplementary tests have generally been made to study the effect of the concentration of detergent.

DIFFUSED-AIR PLANTS

Experiments at sewage works

In Table 1 are shown some results of experiments to measure the performance of a number of diffused-air units equipped with porous diffusers of different types having, when new, normal permeabilities (the flow of air which passes through unit area of the dry diffuser, 1·8 cm thick, under a head equivalent to 5 cm of water) of the order 1·2 l./min cm². All these units were of the ridge-and-furrow type in widespread use in Britain.

The values of the diffused-air coefficient, K_A, range between 0·025 and 0·06 ml/100 ml ppm m, tending to be higher at the outlet than at the inlet end of the aeration tanks presumably owing to the increased purity of the liquid phase at the outlet. It is not possible to give exact figures for the oxygenation capacities of the plants

VOL. 5 1961

TABLE 1. RESULTS OF EXPERIMENTS TO DETERMINE THE DIFFUSED-AIR COEFFICIENT, K_A, AND AERATION EFFICIENCIES OF ACTIVATED-SLUDGE AERATION UNITS AT SEWAGE WORKS

(Approximate flow of air per diffuser, 14–33 l./min. Depth of liquid above diffusers, 3·4–3·6 m.)

Type of aeration unit*	Sampling position (relative to inlet)		Dissolved oxygen (ppm)	Depletion of oxygen from gas (%)	Diffused-air coefficient, K_A, at 10°C (ml/100 ml ppm m)	Calculated oxygenation capacities per diffuser for a flow of 20 l./min (g/hr)	Estimated aeration efficiency (g/kWh)†	
Diffusers	Average distance (m)	Average retention period (approx.) (hr)					Theoretical	Practical‡ (approx.)
	2	0·05	0·6	6·4	0·035	25	1900	1100
Large domes	2	0·05	0·6	6·4	0·037	25	1900	1100
(17·8 cm dia.)	18	0·5	0·6	8·8	0·047	34	2500	1600
at 22·8 cm	160	4·4	6·7	4·5	0·058	42	3100	1900
centres	180	5·0	6·7	4·5	0·058	42	3100	1900
Small domes	2	0·05	0·5	4·5	0·025	18	1300	800
(10·1 cm dia.)	18	0·5	0·5	4·6	0·026	19	1400	800
at 22·8 cm centres	180	5·0	0·6	6·4	0·035	25	1900	1100
Plates	2	0·05	0·6	7·9	0·041	30	2200	1300
20·3 × 15·2 × 1·8 cm	18	0·5	0·6	7·4	0·039	28	2100	1200
22·8 cm between	160	4·4	4·5	7·4	0·061	44	3300	1900
centres	180	5·0	5·8	5·0	0·052	38	2800	1700
Large domes								
(17·8 cm dia.)	2	0·05	0·9	4·8	0·028	20	1500	1000
19·1 cm between centres	180	5·0	5·8	4·8	0·049	36	2600	1600
Small domes								
(10·1 cm dia.)	2	0·05	0·5	6·8	0·038	28	2000	1200
at 22·8 cm	2	0·05	0·5	7·8	0·043	31	2300	1400
centres	15	1	0·5	7·8	0·043	31	2300	1400

* All units were of the ridge-and-furrow type.

† Figures rounded to nearest 100 g/kWh.

‡ Assuming the overall efficiency of the compressor and motor is 0·6.

since the flow of air was not metered, and the values obtained with the hood give only an approximate estimate. The aeration efficiency similarly cannot be determined with absolute accuracy since this also depends on a knowledge of the flow of air and of the efficiency with which the air compressors operate, which is not usually known to an accuracy better than $\pm$ 0·1. The values in Table 1 have been calculated assuming an overall efficiency of 0·6 and for the existing mains pressure at the time of the experiment.

It is seen that the theoretical aeration efficiencies range between 1300 and 3300 g/kWh and the corresponding estimated practical values between about 900 and 1900. Though sufficient data have not been obtained to give an average value of the practical efficiency for British conditions, if the above data are taken as a guide it seems that this is likely to be around 1400–1500 g/kWh.

LABORATORY TESTS

The factors which govern the rates of aeration in diffused-air plants have been elucidat ed largely from experiments in the laboratory. Two types of apparatu have been used. One is a 5000 l. (17·7 ft³) test tank (Fig. 3) which has been used mainly for the measurement of the performance of diffusers by unsteady-state

VOL. 5 1961

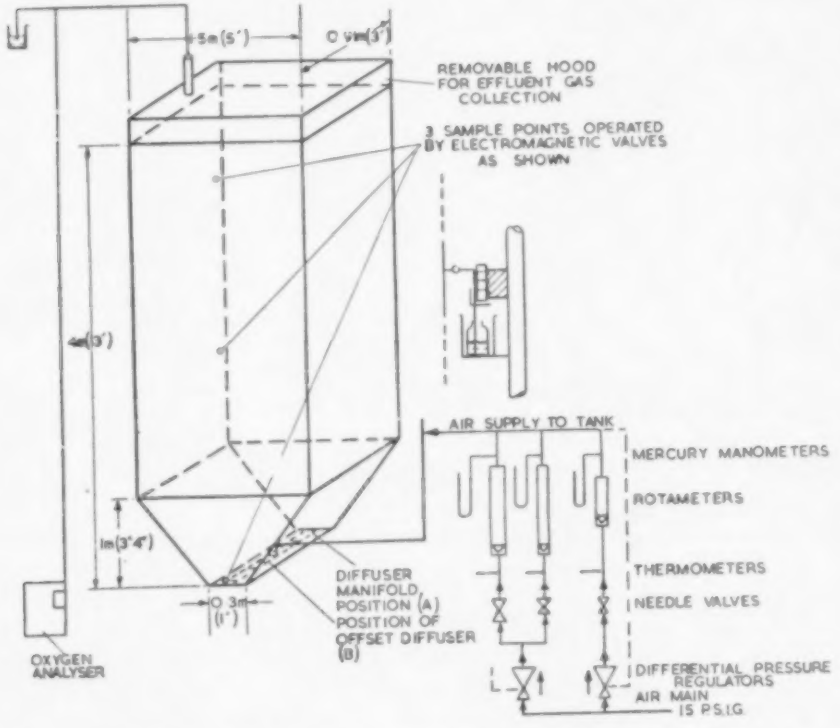


FIG. 3. 5000 l. tank used in the laboratory for investigating the performance of diffused-air systems.

experiments in water containing anionic detergent or in mixed liquor to which the sewage was added on a fill-and-draw basis. The other is a small-scale continuous activated-sludge plant, the aeration unit of which is 4·6 m (15 ft) deep, capacity about 20 l. (0·73 ft³). This has been used mainly to study the effect of the composition of the liquid on the rate of aeration by the steady-state method.

Effect of the composition of the liquid

The degree of purification. In Fig. 4 are shown variations of the value of K_A with period of aeration in a mixture of activated sludge and sewage from a residen-

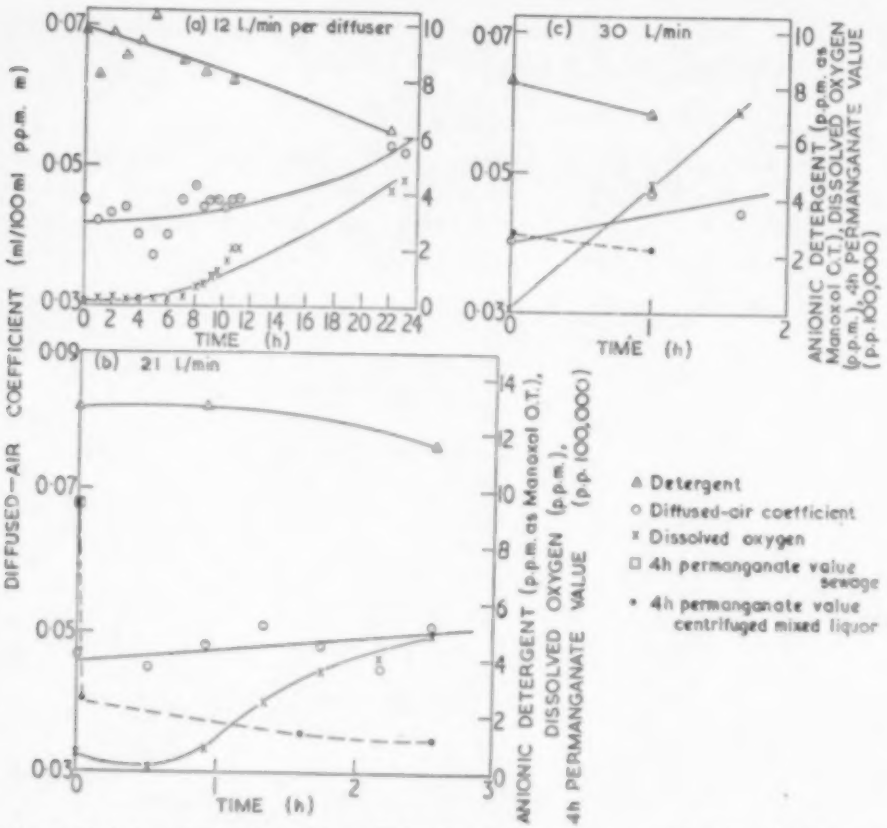

(a) 12 l./min per diffuser
(b) 21 l./min
(c) 30 l./min
DIFFUSED-AIR COEFFICIENT (ml/100ml ppm. m)
ANIONIC DETERGENT (p.p.m. as Manoxol O.T.), DISSOLVED OXYGEN (p.p.m.), 4h PERMANGANATE VALUE (p.p. 100,000)
TIME (h)
Detergent
Diffused-air coefficient
Dissolved oxygen
4h permanganate value sewage
4h permanganate value centrifuged mixed liquor

FIG. 4. Variation of the diffused-air coefficient in mixtures of sewage and activated sludge with the period of aeration and the composition of the liquid phase. Large domes, 3·8 m depth of liquid, 10°C.

tial district in Stevenage New Town, Hertfordshire. The experiments were made in the 5000 l. tank using large dome diffusers with rates of flow of 12, 21, and 30 l./min per diffuser. It is seen that the values apparently increased slightly with the period of aeration or the degree of purification (the rate of which was dependent

on the flow of air) though the scatter of the data in Fig. 4 (b) and (c) is such that the apparent trends may not be significant. The magnitude of the values is of a similar order to those observed at sewage works.

Suspended solids. At the end of the experiment at the lowest flow the solids were allowed to settle and a proportion was removed so as to reduce the concentration

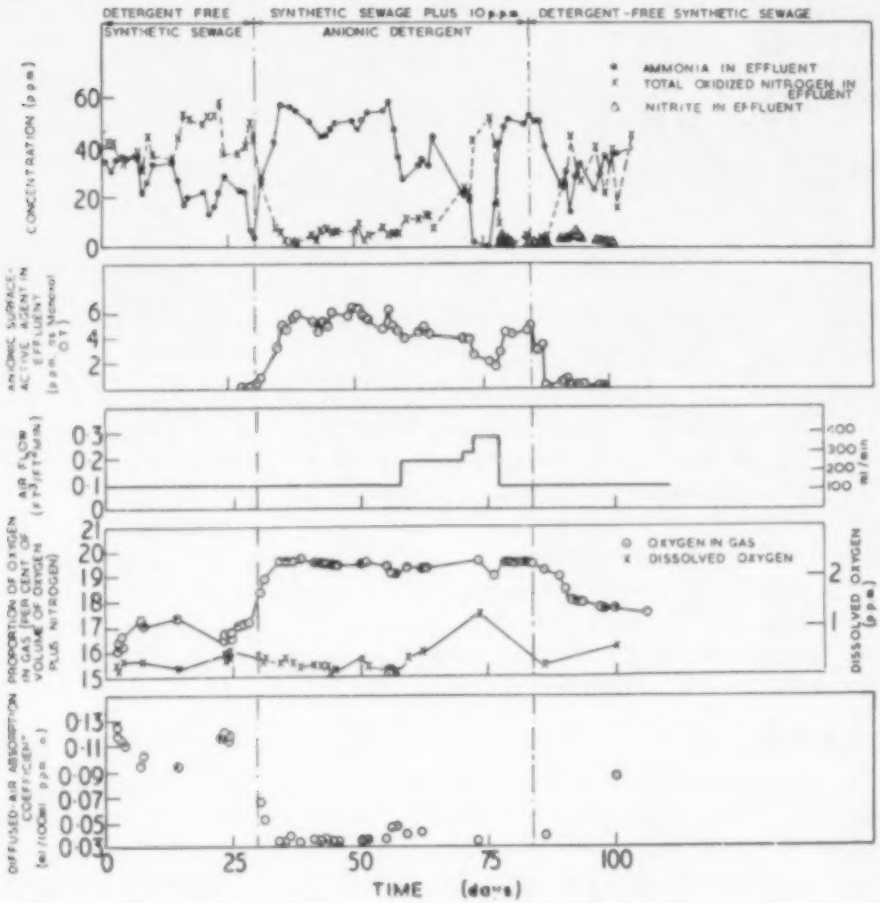

DETERGENT FREE SYNTHETIC SEWAGE
SYNTHETIC SEWAGE PLUS 10 p.p.m. ANIONIC DETERGENT
DETERGENT-FREE SYNTHETIC SEWAGE
AMMONIA IN EFFLUENT
TOTAL OXIDIZED NITROGEN IN EFFLUENT
NITRITE IN EFFLUENT
CONCENTRATION (p.p.m.)
ANIONIC SURFACE-ACTIVE AGENT IN EFFLUENT (p.p.m. as Manoxol OT)
AIR FLOW (c.f.m./sq.ft. of tank)
l./min
PROPORTION OF OXYGEN IN GAS (PER CENT BY VOLUME OF OXYGEN PLUS NITROGEN)
OXYGEN IN GAS
DISSOLVED OXYGEN
DISSOLVED OXYGEN (p.p.m.)
DIFFUSED-AIR ABSORPTION COEFFICIENT* (α/100 at 1 p.p.m.)
TIME (days)

FIG. 5. Effect of anionic detergents on the diffused-air coefficient and on the performance of a laboratory-scale diffused-air activated-sludge plant.

to about 4000 ppm, the relatively small volume of liquid removed being replaced by tap water containing the same concentration of anionic detergent. The liquid was then aerated once more and K_A redetermined. This procedure was then repeated until the suspended solids had been reduced to about 300 ppm. The results indicated that K_A was little affected by suspended solids in the normal range of variation from 1000 to 6000 ppm.

Anionic detergents. In Fig. 5 are shown values of K_A in the small-scale diffused-

air plant and some of the more relevant features of the performance of the plant, in successive periods when the plant was treating a detergent-free synthetic sewage, then the same sewage to which 10 ppm of mixed household detergents had been added, and finally detergent-free sewage again. In Fig. 6 are shown the variations

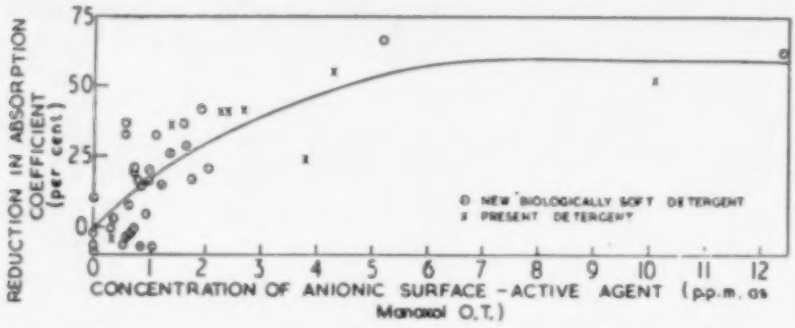

REDUCTION IN ABSORPTION COEFFICIENT (per cent.)
75
50
25
0
0 NEW 'BIOLOGICALLY SOFT' DETERGENT
x PRESENT DETERGENT
0 1 2 3 4 5 6 7 8 9 10 11 12
CONCENTRATION OF ANIONIC SURFACE-ACTIVE AGENT (p.p.m. as Manoxol O.T.)

Fig. 6. Effect of the concentration of detergent in the liquid phase of mixed liquor on the absorption coefficient for oxygen from diffused-air bubbles. Average initial diffused-air coefficient 0·15 ml/100 ml ppm m.

of K_A with the concentration of detergent in the effluent of the plant in similar experiments in which the concentration of detergent in the sewage was varied. From these and similar experiments (7) it was concluded that the value of K_A is reduced considerably by the presence of detergent in the liquid phase of mixed liquor to an extent dependent upon its concentration and that the introduction of detergent-containing sewage will tend to cause a deterioration in the performance of an activated-sludge plant in which the concentration of dissolved oxygen is initially low, whereas the performance of a plant in which the concentration is high may be substantially unaffected.

Effect of flow

The data given in Fig. 4 do not suggest that the value of K_A for porous diffusers is greatly affected by flow of air. This conclusion has been confirmed by experiments in water containing 5 ppm anionic detergent, results of which are given in Fig. 7. It will be seen that, at a given flow, the values of K_A for the four diffusers in a symmetrical arrangement were higher then those obtained with a single diffuser in an offset position (position *B*, Fig. 3). This is presumably due to differences in the motion of the bulk of the liquid in the two cases.

Although the value of K_A was not greatly affected by increasing the flow, the loss of head at the diffusers increased progressively (8). In the case of the plates the head loss increased only from about 15 cm of water at 10 l./min to 30 cm at 100 l./min. The increase for the domes was much greater but this was due in large part to the resistance imposed by an orifice in the pipe through which air is delivered to the inside of the dome, the purpose of which is to ensure an even distribution of air between domes.

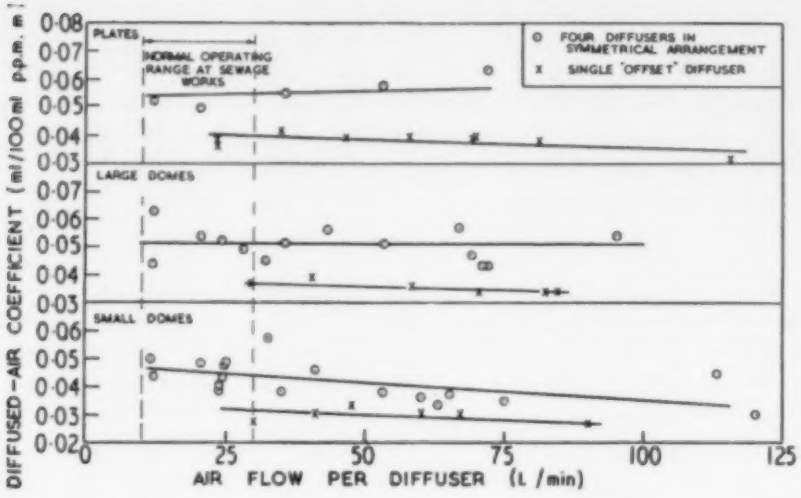


FIG. 7. Effect of the flow of air on the diffused-air coefficient obtained with porous diffusers in water containing 5 ppm anionic detergent. Temperature: 10°C. Depth of water: 3·76–3·87 m.

VOL. 5 1961

Depth

In Fig. 8, curve *A*, is shown the variation of K_A with depth of liquid for large dome diffusers at three different flows within the normal operating range. The product

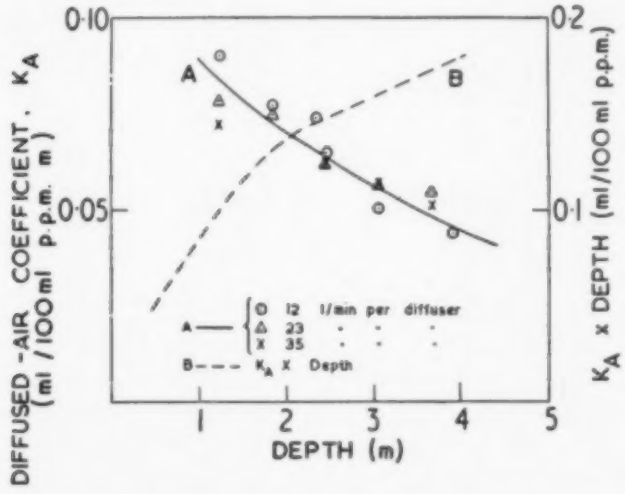


FIG. 8. Effect of the depth of liquid on the diffused-air coefficients and on the volume of oxygen absorbed per unit flow of air during aeration of water containing 5 ppm anionic detergent with porous diffusers of normal permeability. Temperature of liquid: 10°C.

of K_A and the depth, which at a given flow is proportional to the oxygenation capacity of the unit, roughly doubles as the depth increases from 1 to 4 m (curve *B*). However, since the power required to diffuse air from diffusers of normal permeability through a given depth of water is roughly proportional to the depth and to the flow, over quite a wide range (unless the diffuser happens to be heavily clogged), it will be evident that higher aeration efficiencies may be obtained by aerating in shallow rather than deep water.

Aeration at the surface

The trend of curve *B*, Fig. 8, suggests that only a small proportion of the total amount of oxygen absorbed during aeration enters from the surface of the liquid, since if the curve is extrapolated to zero depth the residual value of K_A is small. Measurements of the residual equilibrium concentration of dissolved oxygen maintained in the liquid by absorption of atmospheric oxygen from the surface, when the liquid is de-oxygenated by bubbling nitrogen also support this conclusion (9).

Porosity and permeability

Values of K_A were determined at three different flows for a series of plate diffusers having roughly the same interstitial volume but different pore diameters in the

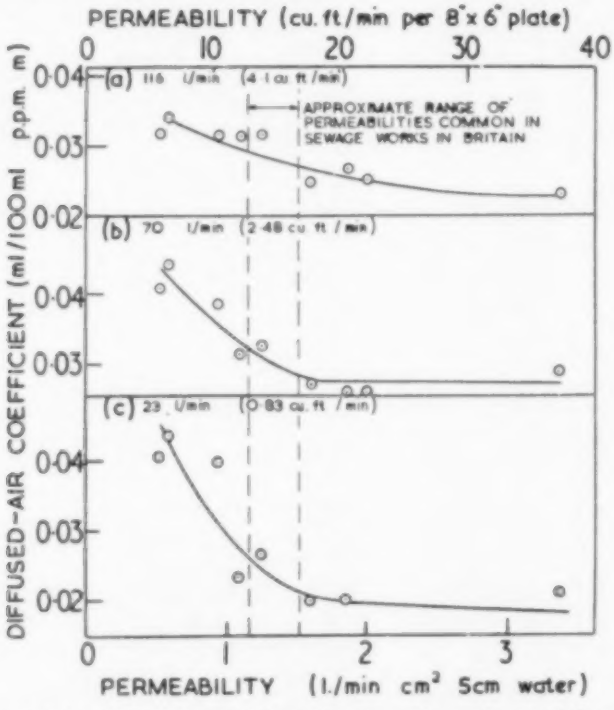


FIG. 9. Variation of the diffused-air coefficient for oxygen in water containing 5 ppm anionic detergent with the permeability of the diffuser. Plate diffusers 1·8 cm (3/4 in.) thick; depth 3·8 m; temperature 10°C.

approximate range 90–300 μ, and with correspondingly different permeabilities (which are known more accurately than the pore size). The plates were mounted in the "offset" position referred to earlier. The results given in Fig. 9 indicate that with diffusers of lower permeability than those normally used at sewage works, more oxygen is absorbed per unit flow. Similar effects have been obtained in clean water by Pöpel (10) and King (11). However, the loss of head at the diffuser increases with decreasing permeability, though even at the lowest permeability and the highest flow the loss was only about 30 cm of water. Thus the power required to diffuse a given flow of air through the depths of liquid normally aerated with diffusers of the lowest permeability tested would not be greatly in excess of that for those normally used when both were in a clean condition. Whether this would still be the case in practice when clogging might be a factor has not yet been investigated.

Aeration by perforated or slotted pipes

In Fig. 10(a) are shown the values of K_A obtained at different flows for a number of perforated pipes, 90 cm (3 ft) long, and for a slotted inverted channel when mount-

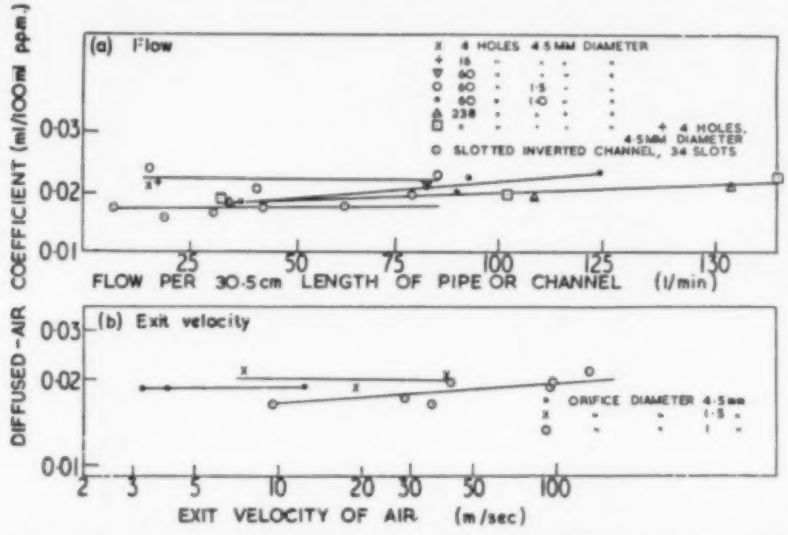


FIG. 10. Variation with the flow or exit velocity of the air of the diffused-air coefficients obtained in water containing 5 ppm anionic detergent with perforated or slotted pipes. Depth of water 3·76–3·87 m. Temperature 10°C. Length of pipe 90 cm.

ed symmetrically in the bottom of the 5000 l. tank. The values are in all cases seen to be lower than those obtained with porous diffusers under comparable conditions and to be not greatly influenced by flow. In Fig. 10(b) the values of K_A for diffusers perforated with holes of uniform size are plotted against the calculated average velocity of the air issuing from them. It appears that variations in the velocity of the air or in the diameter of the orifice within the range 0·1 to 0·45 cm have little influence on K_A.

VOL. 5 1961

Variation of K_A and K_L with bubble size

To provide background data to aid the interpretation of the performance of diffused-air systems, measurements have been made of the values of K_A and K_L with individual streams of approximately uniformally sized bubbles, having average size (expressed as the diameter of the equivalent sphere) ranging from about 0·1 to 1·0 cm in various depths of water in the apparatus shown in Fig. 11. Bubbles were

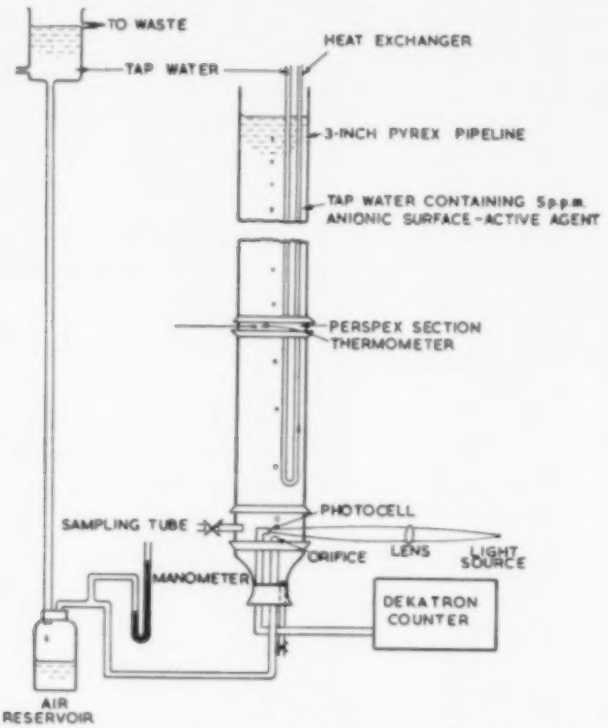


FIG. 11. Apparatus used for determining the variation of the diffused-air and mass-transfer coefficients for oxygen with the size of the bubbles.

diffused into previously de-oxygenated water (containing 5 ppm anionic detergent), at frequencies between about 2 and 15 per sec, by displacing air from the reservoir by a stream of water, through a single orifice of suitable size, the outlet of which was positioned so that the bubbles passed within the light path of a photoelectric bubble counter. Aeration was continued for a sufficient period to add about 2 ppm oxygen to the water. The average bubble size was obtained by dividing the volume displaced from the reservoir by the number registered by the counter and the average velocity by arresting the flow of air at the end of each experiment and timing the ascent of the last bubble in the stream with a stop watch.

The variation of the diffused-air coefficient, K_A, with the size of bubbles and the depth is shown in Fig. 12 together with the corresponding values of K_L for the bubbles.

In determining K_L the rate of absorption of oxygen from the surface was assumed to be negligible by comparison with that from the bubbles. The values of K_L are not greatly influenced by the size of bubbles whereas the values of K_A are approximately

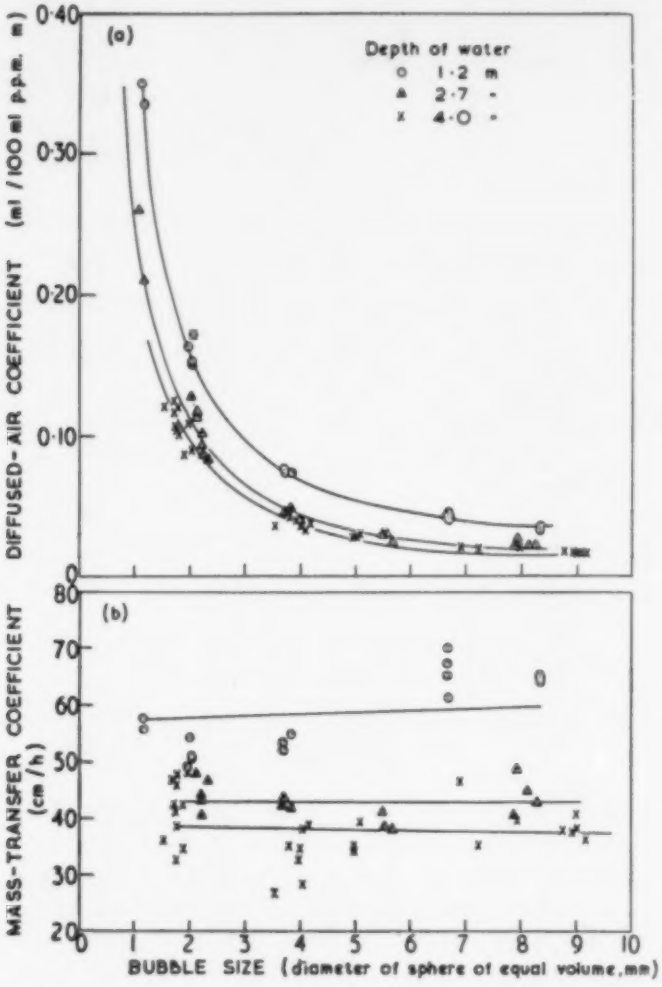


FIG. 12. Variation of the diffused-air coefficient and the overall mass-transfer coefficient with the size of the bubbles. Water containing 5 ppm anionic detergent at 10°C.

inversely proportional to the diameter. Since the velocities of the bubbles do not vary greatly with size in the range studied, the interfacial area per unit volume of air must also be approximately inversely proportional to the bubble size, and this must largely account for the variation in K_A.

Both K_L and K_A increase with decreasing depth in agreement with the results reported above for porous diffusers in the 5000 l. tank, though the values of K_A

VOL. 5 1961

at a given depth are higher than were obtained in the tank. This may be because oxygenated water within the dense swarms of bubbles from the porous diffusers mixes only relatively slowly with the bulk of the liquid.

MECHANICAL AERATORS

There are three main types of mechanical aerator in use in Britain—brush aerators of the type pioneered by Kessener, rotating paddles (12), and aeration cones (13).

Brush aerators and paddles

In Fig. 13 is shown the variation with the depth of immersion of the oxygenation capacity of a full-scale brush-type aerator developed for the regeneration of flax-

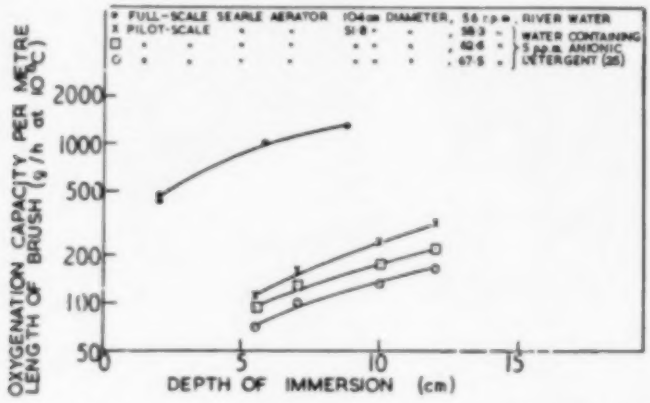

FULL-SCALE SEARLE AERATOR 104 cm DIAMETER, 56 r.p.m., RIVER WATER
PILOT-SCALE " " 58·8 " " 38·3 " } WATER CONTAINING
" " " " " 40·6 " } 5 p.p.m. ANIONIC
" " " " " 67·5 " } DETERGENT (28)
OXYGENATION CAPACITY PER METRE LENGTH OF BRUSH (g/h at 10°C)
2000
1000
500
200
100
50
0
5
10
15
DEPTH OF IMMERSION (cm)

Fig. 13. Variation of the oxygenation capacity of two brush-type aerators with the operating conditions.

retting liquor by Searle (14), and also similar data at different speeds of rotation for a pilot-scale prototype of the same aerator. The results show features typical of most brush and paddle aerators. Over a certain range the oxygenation capacity increases fairly regularly with increasing immersion, and quite rapidly with increasing speed of rotation (usually in proportion to the speed raised to a power between 1·5 and 2·5). Power consumptions tend to vary in a somewhat similar manner. Ultimately of course these trends break down—for instance when the depth of immersion becomes comparable with the length of the teeth, or paddle, or the radius of the "cage", and when the speed becomes excessive. Also the trend of the curves will in general be different for rotors of different design. The same relation would not be expected for a triangular shaped tooth as with a trapezoidal one for instance. Despite these limitations, published results (5, 12, 15, 16) do tend to conform to very rough generalizations of the type shown in Fig. 14 in which the data for paddles have been obtained by extrapolation of the original data (12) to obtain the values at 10 cm. It appears that the ratio of the oxygenation capacity for brush aerators

of diameter 42 cm or greater at any immersion within the range from 5 to 20 cm to that at 10 cm increases roughly from about 0·55 at 5 to 1·9 at 20 cm. Power consumptions vary in a somewhat similar way, the corresponding ratios being 0·6 and 2·1. The deviations of the individual points from the line in Fig. 14 give an idea of the extent to which this rough generalization may be in error.

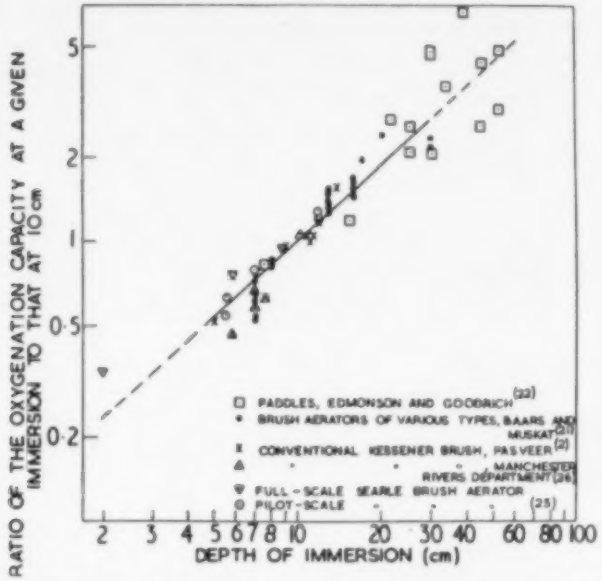

RATIO OF THE OXYGENATION CAPACITY AT A GIVEN IMMERSION TO THAT AT 10 cm
5
2
1
0·5
0·2
2 3 4 5 6 7 8 10 20 30 40 60 80 100
DEPTH OF IMMERSION (cm)
PADDLES, EDMONSON AND GOODRICH (22)
BRUSH AERATORS OF VARIOUS TYPES, BAARS AND MUSKAT (21)
CONVENTIONAL KESSENER BRUSH, PASVEER (2)
MANCHESTER RIVERS DEPARTMENT (26)
FULL-SCALE SEARLE BRUSH AERATOR
PILOT-SCALE (25)

FIG. 14. Variation of oxygenation capacities of brush and paddle aerators with the depth of immersion.

Published figures for the theoretical aeration efficiency of brush aerators operating in water vary between about 1300 and 3500 g/kWh (5, 15, 16); no measurements in mixtures of settled sewage and activated sludge appear to have been reported. However, it has been shown that (in contrast to the effects obtained in diffused-air systems) the oxygenation capacity in water is increased by addition of anionic detergents (1, 17), in some cases very considerably, so that performance in mixtures of detergent-containing sewage and activated sludge may conceivably be higher than in water.

Aeration cones

In Fig. 15 are shown the results of experiments to measure the oxygenation capacities of full-scale high-intensity aeration cones, 1·8 m in diameter, at two sewage works. At one works the aeration tank had a volume of 80,000 Imp. gal and the water aerated contained small concentrations of detergent varying between 0·3 and 2·0 ppm derived from residual dried sludge on the walls of the tanks; at the other the volume of the tank was 78,000 Imp. gal and measurements were made both

VOL. 5 1961

in pure tap water and in tap water to which anionic detergents had been added. Under given conditions the oxygenation capacity increased with increasing concentration of detergent added, the effect being greater at low than at high freeboards (the distance from the lip of the cone to the water surface). Allowing for the different concentrations of detergent in the water the results at the two works were in rea-

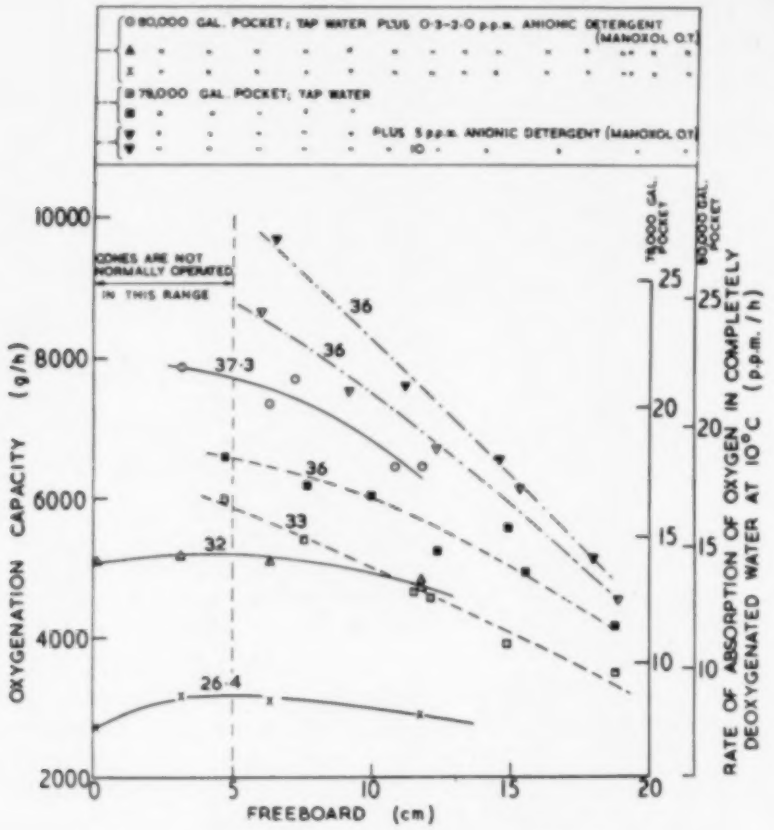


Fig. 15. Oxygenation capacities of aeration cones.

sonable agreement. The oxygenation capacity decreased steadily with increasing freeboard above 5 cm and increased in proportion to the stirring speed raised to a power of about 2·6. Under normal operating conditions the theoretical aeration efficiencies in water were about 1700 g/kWh and in water containing 5 ppm anionic detergent about 1900 g/kWh.

Acknowledgement—This paper is published by permission of the Department of Scientific and Industrial Research, Great Britain.

REFERENCES

1. Downing, A. L., *J. Inst. Publ. Hlth Engrs.* **59**, 80 (1960).
2. Eckenfelder, W. W., *Sewage Industr. Wastes* **31**, 60 (1959).
3. Finn R. K., *Bact. Rev.* **18**, 254 (1954).
4. Walker J. A. H. and Holdsworth, H., *Biochemical Engineering* (edited by Steel, R.) p. 259. Heywood, London (1958).
5. Pasveer, A., *Sewage Industr. Wastes* **25**, 1397 (1953).
6. Longwell J. and Maniece, W. D., *Analyst* **80**, 167 (1955).
7. Downing A. L. and Scragg L. J., *Wat. & Waste Treatm. J.* **7**, 150 (1958).
8. Department of Scientific and Industrial Research, *Water Pollution Research 1958* p. 14. Her Majesty's Stationery Office, London (1959).
9. Department of Scientific and Industrial Research, *Water Pollution Research 1959* p. 7. Her Majesty's Stationery Office, London (1960).
10. Pöpel F., *Proc. Inst. Civ. Engrs.* **14** (1959).
11. King H. R., *Sewage Industr. Wastes* **27**, 894 (1955).
12. Edmondson J. H. and Goodrich S. R., *J. Inst. Sewage Purif.* pt. 2, 17 (1947).
13. Ames Crosta Mills & Co. Ltd., The Simplex High Intensity Aerating Cone, January (1954).
14. Searle G. O., Patent application No. 37140/56, filed 5 December (1956).
15. Baars J. K. and Muskat J., *Zuurstoftoevoer aan water behulp van roterende lichamen.* Report No. 28 of the Research Institute for Public Health Engineering TNO, The Hague (1959).
16. City of Manchester Rivers Department, Report for the year ended 31 March (1958).
17. Baars J. K., *J. Inst. Sewage Purif.* pt. 4, 358 (1955).
18. City of Manchester Rivers Department, Report for the year ended 31 March (1959).

VOL. 5 1961

DISCUSSION

C. L. Siebert, Jr.: In determining aerator efficiencies in grams O_2 per kWh, do you include both agitator and blower horse power on units using both turbine and sparge ring?

In your Fig. 5, the addition of detergent shows a significant increase in the oxygen content of air collected above the tank, but how do you explain the fact that the dissolved oxygen in the tank showed no significant change?

Author: In reply to Dr. Siebert's first query the position was that for systems involving both diffusion of air and mechanical agitation, the total power for the compressor and motor and for the motor driving the impeller were used to obtain what has been referred to as the "practical" aeration efficiency. The "theoretical" aeration efficiencies were calculated from the total net power actually used in aeration after allowing for the efficiencies of compressors and driving motors.

On the second point there was, in fact, a slight decrease, after the addition of detergent, in the average concentration of dissolved oxygen in the aeration unit in the laboratory-scale plant from 0·3 to 0·2 ppm. Exactly how much should be made of these values is, however, uncertain since they are of the same order as the probable accuracy of the analysis, partly because the volume of sample available was small and partly because the analytical procedure itself (the modified Winkler method in which copper sulphate and sulphamic acid are used to suppress biological activity) was not wholly satisfactory. Taking the results as they stand, however, they appear to show that when the concentration of dissolved oxygen fell below 0·3 ppm nitrification was suppressed, but since the estimated percentage reduction in the total oxygen demand due to this was roughly of the same order as the percentage reduction in the rate of solution of oxygen there was not much tendency for the concentration of dissolved oxygen to fall below the value at which nitrification ceased.

K. Wuhrmann: The observation by the authors concerning the entirely different influence of surface active agents in water on the K_L values of compressed air and surface aeration devices

respectively, is in close agreement with our experience. It is remarkable that Kessener and Ribbius (1935) already made the same statement "... it is to be noted that aeration by means of compressed air is influenced far more unfavorably by sewage than is surface aeration", although detergent problems were certainly much less pronounced in 1935 than today.* Normal organic constituents of domestic sewage (e.g., polypeptides, fatty acids, etc.) exert obviously similar effects on aeration to the more recent synthetic surfactants.

AUTHOR: Regarding Dr. Wuhrmann's comment, there was perhaps one additional point to make and that was that in certain instances increases in the rate of aeration had been observed, following the addition of detergents to water aerated by diffused air bubbles, when the bubbles were subjected to large shearing forces at the moment of formation, for instance by pumping a jet of water over the diffuser. In these conditions the size of the bubbles was considerably reduced and it appeared that the resulting increase in the interfacial area outweighed the reduction in mass-transfer coefficient.

* KESSENER, H. J. N. H. and RIBBIUS, F. J., The Institute of Sewage Purification, Annual Summer Conference, Manchester (1935).

FUNDAMENTALS OF OPERATION OF ENTRAINMENT AERATORS

ROY F. WESTON and VERNON T. STACK, JR.
Roy F. Weston, Inc.

THIS paper discusses the development of an aerating device consisting of a flat plate with vertical radial blades, rotating in a horizontal plane. The plate is positioned to rotate at an elevation a few inches below the normal water surface. A drawing of a typical installation is shown in Fig. 1. A photograph of a plant size unit is

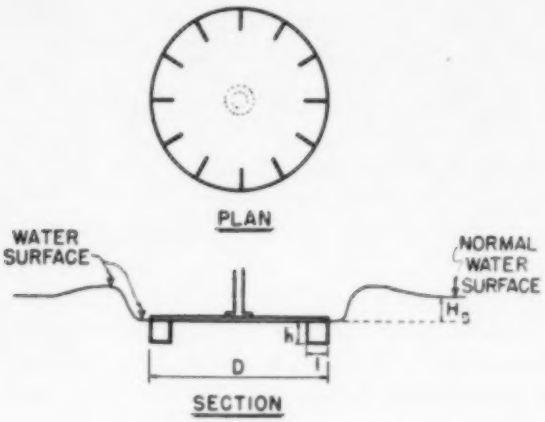


FIG. 1. Entrainment aerator.

shown on Fig. 2. After the type of impeller had been selected pilot plant investigations were conducted. Later a full scale installation was designed and placed in operation.

It is the purpose of this paper to describe briefly the techniques employed to evaluate the performance of pilot and full-scale units and to qualitatively and quantitatively describe the performance characteristics of this type of aerating device.

EXPERIMENTAL PROCEDURES

General

Experiments were conducted to obtain information on oxygen transfer capacity, power requirements and velocity distribution and decay.

Operating conditions were varied to determine the effect of impeller diameter, speed of rotation, depth of submergence, top and bottom blades and geometric proportions. A photograph of a typical impeller is shown in Fig. 3.

VOL. 5 1961

The range in physical test variables is as follows:

(1) Impeller submergence, from 0 to 24 in.
(2) Impeller diameter, 12, 15, 126, 132 in.
(3) Impeller peripheral speed, from 11·5 to 19·0 ft/sec.
(4) Top blade height, from 0 to $2^1/_4$ in.
(5) Bottom blade height, from 0 to $6^1/_2$ in.
(6) Number of blades, 12 and 18.

All the data were obtained by or for the P. H. Glatfelter Company. The Glatfelter pilot plant investigations were conducted in a 100,000 gal. basis (approximately $30 \times 60 \times 9$).

Oxygen transfer and power

Two methods of determining oxygen transfer rates were employed in these studies:

(1) The steady-state biological method.
(2) The non-steady-state method.

The transfer coefficient in the steady-state biological method was determined from the relationship:

$$k_L a = \frac{r}{C_s - C_o} \tag{1}$$

in which $k_L a$ = oxygen transfer coefficient,
r = oxygen uptake rate (ppm/hr),
C_s = saturation concentration of oxygen in waste (95 per cent of that in pure water) (ppm),
C_o = concentration of oxygen in the basin at the test condition (ppm).

Average values were used to determine the oxygen transfer coefficient. Samples were taken for dissolved oxygen determination in the approximate vicinity of the impeller and at midpoints of the basin. At each of these locations both surface and mid-depth samples were taken. Spot checks demonstrated that the dissolved oxygen concentration in the immediate vicinity of the impeller was only slightly greater (0·1–0·2 ppm) than that in the remainder of the basin. From this information, it was concluded that the mixing was substantially complete in both the horizontal and vertical directions. The transfer rate in the wastes was 0·95 that in tap water. The non-steady-state method was run on tap water de-aerated with sodium sulfite using cobalt chloride as a catalyst.

The power calculations were based on amperage, voltage and power factor data with appropriate corrections for mechanical and electrical efficiency.

Analysis of the data

While conducting the experiments it was noted that the power consumption for a given impeller and operating speed decreased as the submergence decreased. At high submergence the plate was covered. The power remained substantially constant. As the submergence decreased a vortex was formed. Further decrease in submergence exposed increasing portions of the impeller plate until the top of the plate was completely exposed except for splash back.

VOL.
5
1961

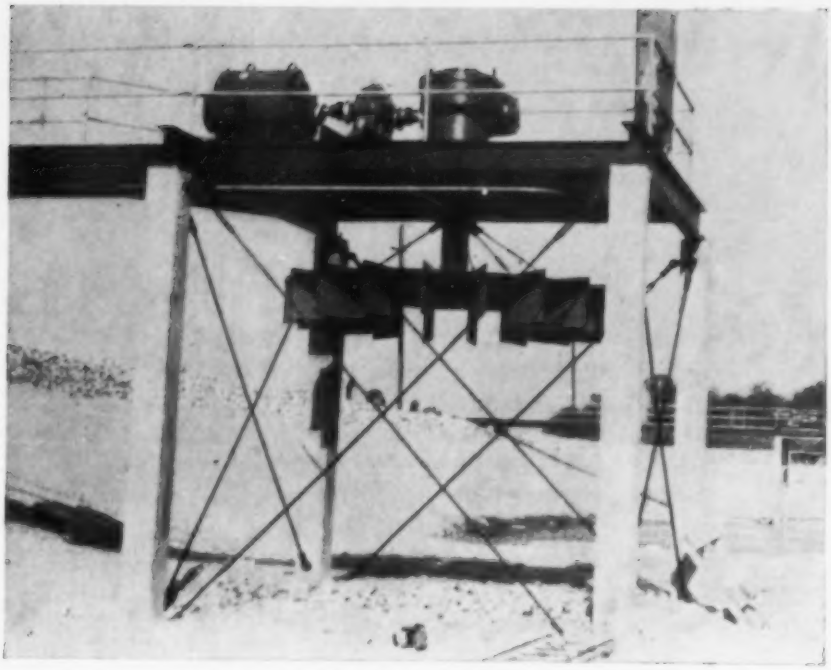


FIG. 2. Entrainment aerator.

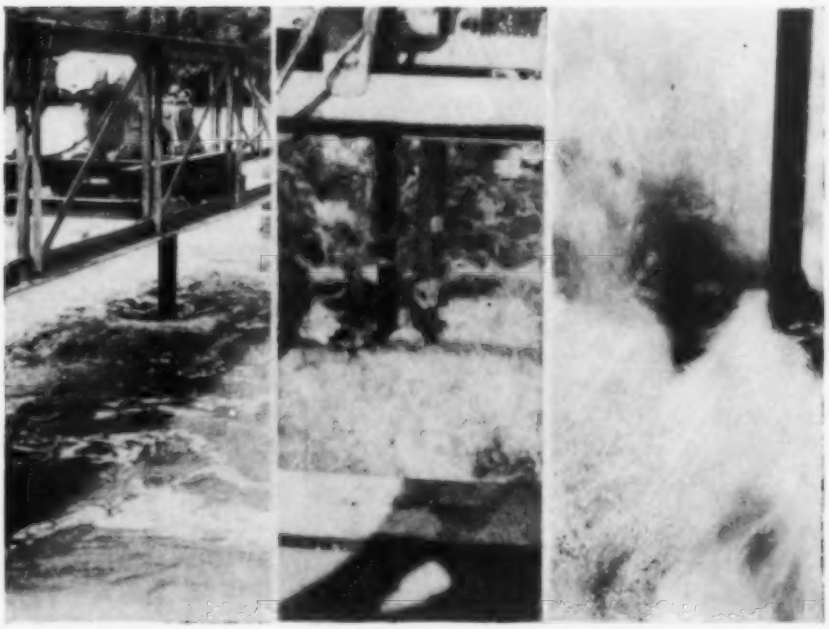


FIG. 3(a), 3(b), 3(c). Entrainment aerator.

It appears that there are three different operating regimes. At high submergence (regime A), a small hydraulic jump is formed, a measurable amount of oxygen is transferred and the power is practically constant. This condition is shown on Fig. 3(a). As the submergence is decreased a second regime occurs (regime B). In this regime the power decreases substantially as the submergence is decreased and the oxygen transfer increases appreciably. This condition is shown on Fig. 3(b).

As the submergence approaches zero, a third regime (regime C) is encountered. The hydraulic jump disappears and gives way to a spraying type of action. Oxygen transfer drops off and power decreases but slightly. This condition is shown in Fig. 3(c).

The different operating regimes may be best seen quantitatively on analysis of the power requirements.

Power

Power variations have been analyzed on the basis of dimensional analysis used by Rushton *et al.* (1).

Power may be expressed as:

$$P_w = N_p N^3 D^5, \tag{2}$$

in which

P_w = horse power transferred to the water,
N_p = dimensionless coefficient referred to as the power number,
N = speed of rotation of impeller (rpm),
D = diameter of impeller (ft).

VOL. 5 1961

The magnitude of the power number N_p is a function of numerous hydraulic and dimensionless parameters of similitude. Laborious and persistent efforts have revealed that the following parameters are significant in the power correlations. These are:

N_F = Froude Number $\dfrac{DN^2}{g}$

$\dfrac{H_s+h}{H_s+h+H_v}$ = ratio of static to dynamic head,

in which

H_s = submergence head (ft),
h = height of blade,
H_v = $V_p^2/2g$ = peripheral velocity head in which $V_p = \pi DN/60$,
l/D = blade length to diameter ratio,
h/D = blade height to diameter ratio,
n/n_8 = ratio of number of blades, n, to a reference number of blades, 8,
D/D_1 = ratio of impeller diameter D to reference diameter of 1·0 ft.

The power requirements for any given impeller and operating condition may be calculated from the following formulae:

$$P_w = C_1 N^3 D^5 N_p a \left[\frac{H_s+h}{H_s+h+H_v}\right]^b (1/D)^c (h/D)^d (n/n_s)^{0.7} (D/D_1)^f, \tag{3}$$

and

$$N_p = C_1 N_p a \left[\frac{H_s+h}{H_s+h+H_v}\right]^b (1/D)^c (h/D)^d (n/8)^{0.7} (D/D_1)^f. \tag{4}$$

The exponents a, b, c, d and f and the constant C_1 must be determined from experimental data. The exponent 0·7 was taken from Rushton's (1) data because only one number of blades was used for each diameter of impeller. Therefore, there was no way of differentiating the effect of the number of blades from the available data.

The relationship between the actual and calculated N_p values for the different experimental and test runs is shown in Fig. 4.

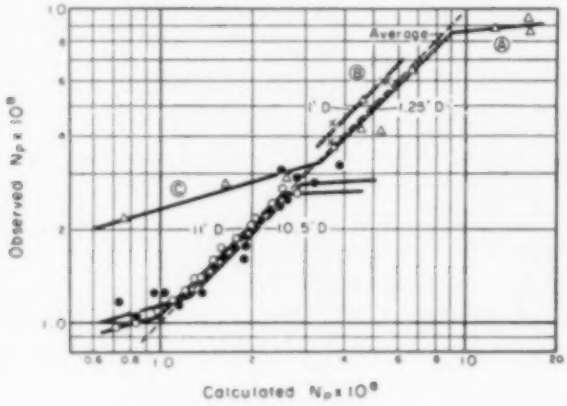


FIG. 4. Horse power correlation.

It is apparent that reasonably good correlation between small and large scale units is possible for regime B using the above relationships. The limiting power numbers for each operating regime must be determined experimentally for each turbine.

The three different regimes are clearly indicated.

Oxygen transfer

Observations of aerator operation and an analysis of the horse power relationships indicate that oxygen is transferred by three different mechanisms. These are:

(1) Entrainment due to turbulence in the hydraulic jump.

(2) Air entrainment by the impeller.

(3) Transfer due to surface turbulence or interchange.

The first two of these mechanisms can be differentiated. Practically, the third is an indistinguishable part of the two entrainment mechanisms.

In order to estimate the quantity of oxygen transfer for different impellers and operating conditions, it is necessary to postulate the mechanism of transfer.

The magnitude of transfer in the hydraulic jump may be estimated from the high submergence data. Kalinske (2) has shown that air is entrained in a hydraulic jump and that the amount of entrainment is a function of the head lost in the jump and the rate of flow. In this case it may be assumed that oxygen transfer is proportional to the power loss in the jump. The principal physical condition controlling the characteristics of the jump is the depth from the normal water surface to the bottom of the impeller blade. It is assumed that:

$$W_{o,j} = \frac{C_2 P_w}{H_s + h} = \frac{C_2 N_p N^3 D^5}{H_s + h}, \tag{5}$$

in which

$W_{o,j}$ = oxygen transfer in the hydraulic jump at 20°C and 0·0 ppm dissolved oxygen concentration in the body of the solution.

C_2 = constant.

This equation is dimensionally correct. It accounts for all oxygen transfer for regime A and part of the oxygen tranfer in regimes B and C.

The decrease in horse power with decrease in submergence and increase in oxygen transfer can best be accounted for by air entrainment by the impeller.

This is possible because a low pressure area should exist behind each blade. Atmospheric pressure should exist at the top outside edge of each blade. Thus entrainment is conceivable. Observations bear out this assumption.

If such a condition exists, it would interfere with the pumping capacity of the impeller. Therefore, the power required to rotate the impeller would decrease as the air entrainment increased. The actual total horse power would be equal to

$$P_t = \frac{Q_t w_t H_t}{C_3}. \tag{6}$$

It can be seen from Fig. 1 that atmospheric pressure exists at the periphery of the impeller. Therefore, the submergence head (H_s) as well as the head imparted by the rotation of the impeller (H_v) is converted to velocity head.

Equation (6) may be expressed as:

$$P_t = \frac{\pi D h V_t}{C_3} \quad p w_w \quad p \frac{V_t^2}{2g}, \tag{7}$$

in which

$V_t = \sqrt{2g}\ H_t = f_1 \sqrt{[2g\ (Hs) + Hv)]}$,

f_1 = ratio of the actual velocity V_t to the theoretical velocity $\sqrt{[2g\,(Hs + Hv)]}$,

$p = \frac{w_t}{w_w}$ = ratio of the actual weight of the mixture being pumped to the weight of water in lb/ft³.

VOL. 5 1961

Since

$$P_w = P_i = N_p N^3 D^5$$

It can be shown that:

$$f_1 = \sqrt[3]{\left[1{\cdot}2584 \times 10^8 D/h \frac{N_p}{p^2(1+Hs/Hv)^{1{\cdot}5}}\right]}. \tag{8}$$

Observations on pressure differences due to objects moving through water indicate that the head differential behind an impeller blade should be:

$H_d = C_4 V^2/2g$ (assuming water flow alone causes this differential).

If it is assumed that air is entrained, the quantity entrained should be proportional to the product of an area and a velocity. The area should be proportional to the peripheral length of the impeller, the number of blades, height of blades and other geometry parameters.

The velocity should be a function of H_d. Under the conditions of operation the efficiency of transfer of oxygen from entrained air would be practically a constant over a wide range of operating conditions.

Therefore, for any given impeller:

$$W_{o,e} = C_5 w_a D^3 h N f_1 V/(1+Hs/Hv). \tag{9}$$

Analyses of available oxygen transfer data for a number of impellers indicate that:

$$W_{o,e} = C_6 w_a D^3 h N f_1 V/(1+Hs/Hv)(h/D)^h (1/D)^g (n_1/n^1)^j \tag{10}$$

in which the exponents g, h and j and the constant C_6 are determined by experimental observations.

$W_{o,e}$ = oxygen transfer by impeller entrainment at 20°C and 0·0 ppm dissloved oxygen concentration in the body of the solution.

$\dfrac{n_1/n^1 = \dfrac{n/D}{n_1/D_1}}{}$ = ratio of number of blades per unit of diameter for a given impeller to the number of blades per unit of diameter for a reference impeller.

w_a = weight of air (lb/ft^3) at atmospheric conditions.

The above equation is dimensionally correct. Obviously, the quantity of oxygen transferred under given conditions is a function of temperature and other physical and chemical factors influencing the efficiency of transfer. The equation applies to regime B only. Corrections must be applied accordingly.

The total quantity of oxygen transferred is:

$$W_{o,t} = W_{o,i} + W_{o,e}. \tag{11}$$

Therefore:

$$W_{o,t} = \frac{C_2 N_p N^3 D^5}{H_s + h} + C_6 w_a D^3 h N f_1 V(1+H_s/H_v)(d/D)^a (1/D)^h (n_1/n^1)^j. \tag{12}$$

Correlations between observed and calculated oxygen transfer quantities are shown in Fig. 5. These correlations have been corrected to pure water conditions, 20°C and 0·0 ppm dissolved oxygen in the aerated solution. Equation (12) is reliable for regime B only.

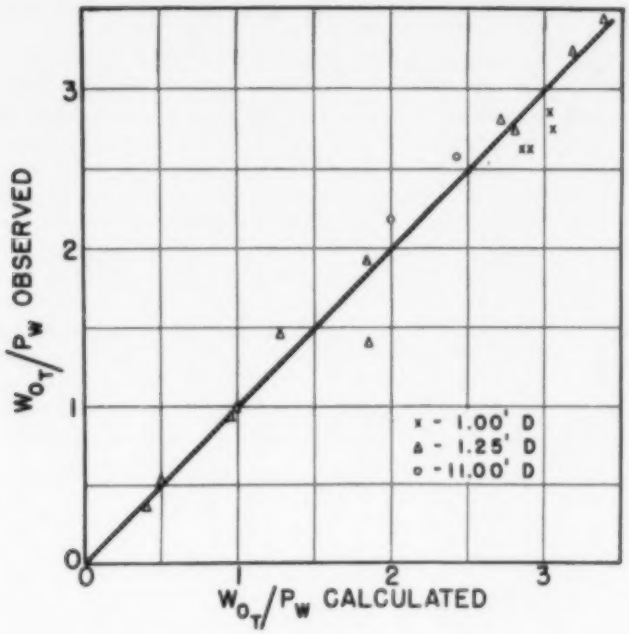


FIG. 5. Oxygen transfer correlation.

SUMMARY

The above analysis of the operating characteristics of the entrainment aerator indicates the possibility of this type unit being a simple, effective and efficient aerating device. The transfer of 3 to 4 or more lb/hr of oxygen per unit of horse power transferred to the water appears to be entirely practical. This should be possible for units varying in size from less than 1 to more than 150 lb of oxygen transfer per hour.

Acknowledgement—The assistance of John F. H. Walker in analyzing and correlating the test data is acknowledged. It is greatly appreciated that the P. H. Glatfelter Company granted permission to publish data obtained by them or on their behalf.

REFERENCES

1. RUSHTON, J. H., COSTICH, E. W. and EVERETT, H. J., Power characteristics of mixing impellers. *Chem. Engng. Progr.* **46**, 8, 9 (1950).
2. KALNSKE, A. A. and ROBERTSON, J. M., Air entrainment-closed conduit flow. *Trans. Amer. Soc. Civ. Engng.* **108**, 1435–47 (1943).

VOL. 5 1961

DISCUSSION

JAMES J. MCKEOWN: (1) Is there any information available on density changes due to increased air entrainment as it effects power requirements? (2) Is equation (2) dependent on the type of impeller blade used? (i.e. Would slotting or perforating the blades alter the speed and diameter exponents and would a six-bladed impeller yield the same exponents as an eight-bladed one? (3) Was any increase noted in the oxygenation capacity of the turbine upon the introduction of surface active agents?

AUTHOR: (1) The variations in horse power shown in Fig. 4 are due primarily to the effects of air entrainment. The effect of density variations alone can be obtained from equation (8). In practical experience it was found that most of the density variations were less than 5 per cent.

(2) The only factor in equation (2) influenced by the type of blade being used would be power number N_P. Obviously any modification of the impeller blade that would alter its pumping capacity would change the exponents on the significant dimensionless parameters used to characterize the impeller being used. The number of blades definitely influence the power requirements of the impeller. The effect of the number of blades on the power number is shown in equation (4). In view of the fact that the power number, N_P, is dimensionless, equation (2) is dimensionally correct as shown. Therefore, no blade modification would alter these exponents. Such modifications would only alter the magnitude of the power number.

(3) No experiments were conducted to determine the importance of the addition of a surface active agent. Nevertheless, experiments were conducted on materials which contained surface active agents. It appeared that surface active agents had no adverse effect on oxygen transfer capacity, and may have had a slight advantageous effect in some cases.

POWER CONSUMPTION FOR OXYGENATION AND MIXING

A. A. KALINSKE

Infilco Incorporated

THE fundamental principles of fluid mechanics, as they apply to mixing, will be presented in this paper and analyzed so far as power requirements are concerned. The controlling factors relating to oxygen absorption from air by liquids will be studied in order to make comparisons, so far as energy requirements are concerned, between various methods for getting oxygen into liquids. Comparisons will be made in regard to mixing and oxygenation by means of compressed air and by various mechanical devices and methods.

VOL. 5 1961

A. FLUID MECHANICS OF MIXING

Mixing in fluids is a turbulence diffusion phenomena and can, therefore, be analyzed using the known principles relating to transfer of fluid properties, and suspended or soluble materials, in a fluid mass. In addition, the energy requirements to accomplish the desired mixing can be studied by utilizing the basic facts relating to energy dissipation in turbulent fluids.

The principles of diffusion in turbulent fluids are well understood and have been presented by the author (1, 2), Orlob (3), and others. The phenomenon is frequently referred to as "eddy diffusion", and can be characterized by a so-called eddy diffusion coefficient. This coefficient is directly proportional to the "intensity" of the turbulence and the so-called "scale" of the eddies, or their statistical average size. The speed of diffusion is proportional to this coefficient and it can be increased by either increasing the "intensity", which is characterized by a statistical average of the velocity fluctuation, or by increasing the size of the eddies, or by increasing both. From a practical standpoint the magnitude of the velocity fluctuations can only be increased by supplying more energy to the fluid, or by increasing the "energy of turbulence". The factors controlling the dissipation of this energy are well understood (4), and without going into great technical detail, suffice it to say, that viscosity and the size of the smallest eddies are the controlling factors. The large eddies which are important in the diffusion process are relatively unimportant so far as energy dissipation is concerned. The production of small, intense eddies greatly speeds up the rate of fluid energy dissipation into heat.

Since the magnitude or intensity of turbulence involves solely the putting in of energy into a mass of fluid, Orlob (3), derived from the principles of dynamic similarity the relation between the eddy diffusion coefficient D_t, the rate of energy

dissipation, E, per unit mass and the scale of the eddies, L, participating in turbulence diffusion:

$$D_t = kE^{1/3}L^{4/3}, \tag{1}$$

where

k = proportionality constant.

The practical significance of this relation is as follows: Turbulence, diffusion or mixing can be increased by adding energy to the fluid mass and by producing large scale eddies. However, it is important that large scale eddies be produced, since the mere adding of energy is of relative small significance, unless, at the same time, large-scale eddies result. This basic principle can now serve as a guide in designing mixing equipment.

It normally is impractical and uneconomical to fill an entire liquid mass, say a basin, with turbulence producing devices, such as for example, various types of stirrers or paddles. Equipment that produces localized turbulence, such as stirrers, is satisfactory for small fluid masses, but cannot produce uniform turbulence in a large mass unless the geometric proportions of the stirrers are comparable to the basin, or high intensity turbulence is produced, which as we note from equation

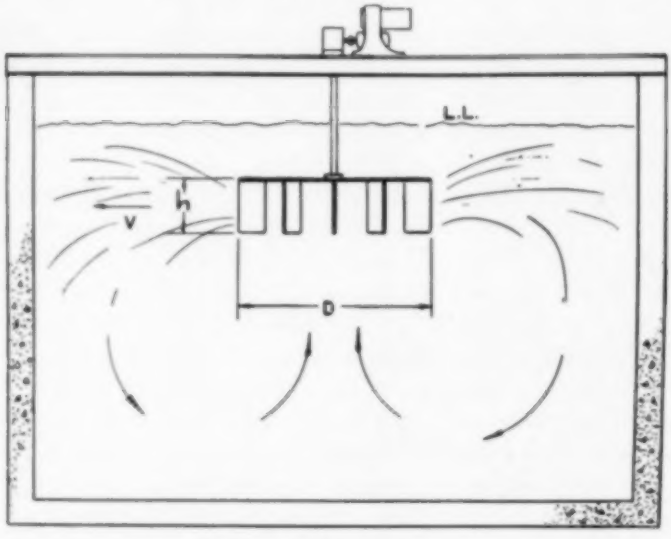


FIG. 1. Turbine type mixer.

(1) is not an economical way for getting good diffusion. The most common method used in present-day mixing operations is to install in a basin a turbine or propeller which pumps a large quantity of liquid. This stream issuing from the pumping device is directed and controlled, in a three- dimensional manner, so as to penetrate the remote points of the liquid mass. This stream, as it moves into a mass of liquid

generates turbulence, and the intensity and scale of this turbulence will be proportional to the velocity of the stream and its relative size.

A typical mixer, of the turbine type, is shown in Fig. 1. The pumpage and issuing stream from the turbine periphery can be analyzed using the basic principles of diffusion of submerged jets (5). The authors of the paper given in reference (5) analyze theoretically, and verify experimentally, the characteristics of streams issuing into a mass of fluid. They present a generalized relation between the initial velocity of the stream and the centerline velocity, for various distances downstream, as the stream penetrates a fluid mass:

$$V_x/V_0 = 2{\cdot}28\sqrt{\left(\frac{B_0}{x}\right)}, \tag{2}$$

where V_x = centerline velocity at distance x downstream,

V_0 = issuing initial velocity,

B_0 = height of issuing two-dimensional stream.

Since the flow, Q_0, per unit width of stream, equals V_0B_0, equation (2) can be written:

$$V_x = 2{\cdot}28\sqrt{\left(\frac{V_0Q}{x}\right)}. \tag{3}$$

The power, P, that must be applied to a pumping machine, such as the turbine mixer shown in Fig. 1, can be written as:

$$P = KQ_0V_0^2, \tag{4}$$

where K = proportionality constant.

The effectiveness of a mixer is related to its pumping capacity and the distance that the liquid it pumps penetrates into the liquid mass. Thus, for any distance, x, from the mixer, we can reason on the basis of equation (3) that the magnitude of the remaining velocity in the issuing stream, V_x, will vary directly as the one-half power of the initial velocity and one-half power of the mixer discharge. This relation has tremendous significance when studied in the light of the power relation, equation (4). For example, if we double V_0, without changing Q_0 (by reducing B_0), we increase V_x by 1·4 times, but we increase the required power by 4 times. However, if we double Q_0, without changing V_0 (by increasing B_0), we again increase V_x by 1·4 times, but we only increase the power by 2 times. We thus see the great advantage in using larger streams of lower velocity. In addition, by increasing the size of a stream we increase the scale of the turbulence which in itself is very important, according to equation (1), in increasing diffusivity.

From a practical standpoint, the above analyses show the great advantage, from power consumption considerations, in using slower moving pumping and mixing units in order to obtain the desired mixing and diffusion. Of course, one thing to keep in mind, so far as keeping settleable material in proper suspension is concerned, is that there is some minimum velocity which must be maintained in all parts of a mixed basin to prevent settling out of this material. However, it

VOL. 5 1961

is not economical from a power consumption standpoint to maintain any higher velocities than are actually needed, since good mixing and high velocities are not necessarily synonymous. We have established that for properly designed turbine type mixers it is possible to mix and keep in suspension the organic solids in sewage sludge digesters, and in activated sludge basins, with the expenditure of from $\frac{1}{4}$ to $\frac{1}{2}$ h.p. per 1000 ft^3 of liquid volume.

B. MIXING WITH AIR OR GASES

Mixing is frequently accomplished by bubbling air, or other gases into liquids. This can be done in a controlled fashion, by use of a so-called submerged riser pipe, or by bubbling the air through small openings or diffusers without any confinement. A more efficient pumping action will be obtained if the former arrangement is used, since there will be less "slip" between the gas bubbles and the liquid can be directed in a preferred manner. A schematic arrangement for such a gas or air-lift circulator is shown in Fig. 2.

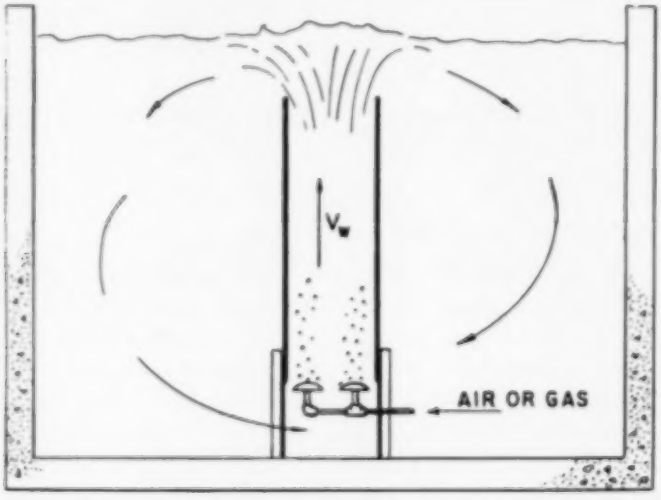


FIG. 2. Gas-lift circulator.

In order to compare properly such air or gas mixing with mechanical mixing, it is necessary to compare power requirements for equal liquid movement or pumpage. First, it is necessary to determine the power required to compress the gas so that it can be forced into the liquid in order to overcome the hydrostatic head and any pressure losses through pipes and diffusers. Taking into account the inherent losses in adiabatic compression the power required is given by the following equation:

$$P = 0{\cdot}28 Q_a \left[\left(\frac{p_2}{p_a} \right)^{0.28} - 1 \right] \tag{5}$$

= horse power,

where Q_a = ft³/min of air under standard conditions,

p_a = absolute atmospheric pressure,

p_2 = absolute pressure of compression.

There have been various relations developed for expressing the rate of circulation (or pumpage) for air-lifts in terms of air flow, etc. One of the earliest studies made (6) proposed use of either the head relation or the energy equation as a starting point. The author used the energy relation in connection with a study of steam lifts (7). A recent study, devoted specifically to gas-lift circulator design (8), uses the head relationship. However, the energy relation is somewhat simpler, and we will use it in our analyses. The energy relation is based on the premise that the energy in the compressed air or gas will be used up in accelerating the liquid to a velocity, V_w, and overcoming any frictional resistances. After some simplification this relation becomes:

$$32\left(\frac{Q_a}{Q_w}\right)\ln\left(\frac{p'_2}{p_a}\right) = C\frac{V_w^2}{2_g}, \tag{6}$$

where Q_w = liquid pumped (ft³/min),

ln = logarithm to base e,

C = coefficient, greater than unity,

p'_2 = pressure of gas as it enters liquid.

The magnitude of C will depend on the frictional resistances. For an arrangement such as shown in Fig. 2, a value of 1·5 is reasonable. The difference between p_2 and p'_2 is the head loss due to friction as the gas comes from compressor into liquid, and includes pipe friction and diffuser losses. For the present, let us consider these as minor and neglect them in our analyses.

By combining equations (5) and (6) a relation is obtained which gives the liquid pumpage in terms of power used to compress the gas, and the gas pressure.

$$Q_w = \frac{115P}{YZH_m}, \tag{7}$$

in which $Y = \left(\frac{p_2}{p_a}\right)^{0\cdot28} - 1$,

$Z = C/\ln\left(\frac{p_2}{p_a}\right)$,

$H_m = V_m^2/2g$,

V_m = mean liquid velocity (ft/sec),

g = 32·2.

Assuming a submergence for the gas entrance of 15 ft. and atmospheric pressure

VOL. 5 1961

of 14·7 lb/in², taking $C = 1{\cdot}5$, and assuming a value for V_w of 6 ft per sec, which is fairly reasonable, though somewhat on the low side, we have:

$$Q_w = 435P. \tag{8}$$

For turbine mixers the power relation can be written as:

$$P = KQ_w V_p^2, \tag{9}$$

where V_p = peripheral speed of turbine (ft/sec).

For certain type turbines, operating in water-like liquids, the coefficient K has been determined as equal to about $0{\cdot}376 \times 10^{-4}$. Using a value of $V_p = 4$ ft per sec, a value normally used for mixing operations, we have:

$$Q_p = 1700\ P \tag{10}$$

Thus it is seen that for a comparable pumpage or circulation rate, from a power consumption standpoint, the direct mechanical circulator is several times more efficient. Therefore, any degree of mixing and diffusion, can be accomplished for a considerably smaller expenditure of energy by using a proper type of mechanical mixer than by use of a so-called gas or air-lift agitation or circulation scheme.

C. OXYGENATION USING COMPRESSED AIR

The theory of solution of oxygen from air bubbles has been fairly well established and there is no need to review it in any detail. Several excellent papers covering this subject are contained in the *Proceedings of the First Manhattan College Conference on Biological Treatment of Sewage and Industrial Wastes* (9). Of utmost importance is bubble size and mixing, or turbulence diffusion. So far as aeration of activated sludge is concerned, the important factor that must be evaluated in comparing various schemes of supplying oxygen, is power needed per pound of oxygen dissolved in unit time. First cost of the necessary equipment and basins must also be studied, but that is not subject to general analysis, since such costs will depend on many factors peculiar to any particular installation.

Using equation (5) for power required to compress air we have the relation:

$$W_0 = \frac{3{\cdot}43E}{Y}, \tag{11}$$

$$= \text{lb } O_2 \text{ absorbed per h.p. per hr,}$$

where E = efficiency of absorption,

Y = (see equation 7).

It is known that with porous diffusers normally used in activated sludge aeration basins, the E varies between about 0·05 to 0·08. For $E = 0{\cdot}06$ and a pressure of 6·5 lb/in², equation (11) gives for W_0 the value 1·88. The oxygen absorption efficiencies used in this paper relate to oxygenation of activated sludge at a temperature of 85°F or less, when maintaining a dissolved oxygen of at least 2 ppm, and with the oxygen absorbing characteristics of the liquid being 95 per cent or greater that of clean water.

D. OXYGENATION WITH COMPRESSED AIR AND MECHANICAL DISPERSERS

In order to increase the absorption efficiency, eliminate maintenance problems associated with porous diffusers, and provide more flexibility of operation, mechanical dispersers are being used more and more widely for aeration of activated sludge.

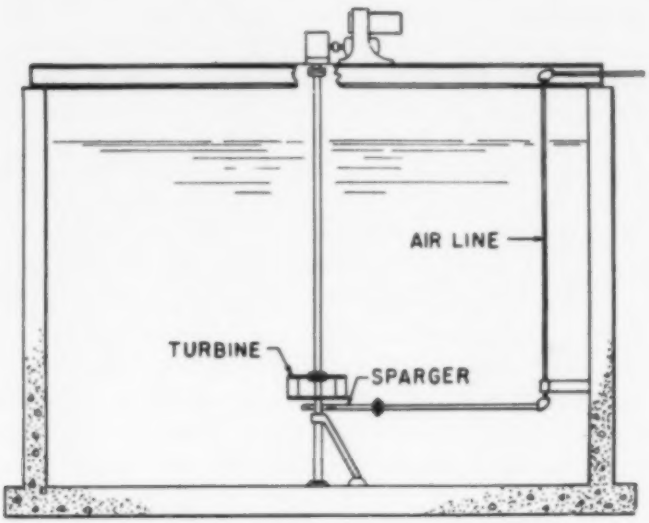


FIG. 3. Mechanical disperser for compressed air.

Typical equipment of this type is shown in Fig. 3, and design data has been presented by the author in a previous paper (9). Because of the high degree of mixing that such a mechanical disperser provides, absorption efficiencies of up to 25 per cent and higher can be obtained. The exact efficiency that can be obtained depends on the peripheral speed of the turbine and the air loading. By using lower air loadings, higher efficiencies can be obtained, with the result that more of the total power is put into the liquid by the turbine and less power is used for compressing the air. This results in an overall reduction in power required per unit weight of oxygen dissolved.

The total power needed per pound of oxygen dissolved per hr is given by:

$$W_0 = \frac{0{\cdot}96E}{0{\cdot}28Y + p_t}, \tag{12}$$

where E = oxygen absorption efficiency for turbine gas disperser,

Y = (see equation 7),

p_t = horse power needed by disperser per ft^3 of air, to attain absorption efficiency E.

VOL. 5 1961

Our design data for turbine air dispersers indicates that for aeration of activated sludge, under conditions specified in Section C, the value of p_t when $E = 0{\cdot}25$ is 0·056. The value of W_0 from equation (12) is then 2·7 lb of O_2 per h.p. per hr. For a value of E of 0·12 we use a value for p_t of 0·022, which gives for W_0 the value of 2·2.

Comparing the above values of W_0 with the value obtained in Section C, we see that oxygenation using compressed air is less efficient than when a portion of the power is supplied directly to the liquid, without the intermediate step of compression of the air. In general, the smaller we can keep the portion of power used for compression, the more efficient will our oxygenation be, so far as power per pound of oxygen dissolved is concerned. The only limitation to this is that when we attempt to obtain very high oxygen absorption efficiencies, say above 25 per cent, the power used by the turbine increases rapidly and such increase in power is not off-set by a sufficient decrease in power for compression of air.

Data indicates that the value of E is not significantly decreased by using lower submergences when mechanical dispersers are employed. Thus a very significant reduction in power of air compression can be obtained by locating the dispersers at smaller submergences than would be needed with non-mechanical diffusers. The mixing that is obtained is largely independent of the amount of air, and thus suspensions of solids can be maintained even if the necessary oxygenation is quite low. This is not true when nonmechanical diffusers are employed.

The above analysis of power consumption leads one to the conclusion that perhaps the most efficient oxygenation means would be where all the power was applied directly to the liquid, mechanically, without the utilization of compressed air at all.

E. OXYGENATION BY MECHANICAL SURFACE AERATORS

In line with the conclusions arrived at in Section D, investigations were made to design a mechanical surface aerator whose primary function would be to supply oxygen to the liquid in as efficient a manner as possible. In order to approach the design of such equipment scientifically, reference was made to the literature relating to surface aeration of water flowing at relatively high velocities. Some very basic studies have been made on this interesting hydraulic phenomena at the University of Minnesota by Dr. L. G. Straub (10). The conclusion is arrived at that aeration depends primarily on the turbulence intensity of the flow.

It appeared that a rational design could be made for a mechanical surface aerator if it was designed on the basic idea of establishing a radial flow, and knowing the characteristics of this flow the air entrainment could be related to these characteristics expressed quantitatively. For a long time hydraulic engineers have known about the excellent mixing and air entraining properties of the so-called hydraulic jump occurring in supercritical open channel flow. However, no quantitative data was found which related all the variables entering into this phenomenon, which would permit establishing a design relationship.

In 1943, the author was in charge of an extensive basic research study at the State University of Iowa on air entrainment of hydraulic jumps occurring in partially full closed conduits. By making this study in closed conduits it was possible to measure the quantity of air entrained and relate it to the rate of water flow and the characteristics of the hydraulic jump. In order that a hydraulic jump occur, the free surface flow must be at a so-called supercritical velocity, or quantitatively stated, the Froude number of the flow must be greater than unity. The Froude number is defined as:

$$F = V_1/\sqrt{(gy_1)}, \tag{13}$$

where V_1 = initial water velocity (ft/sec),

g = 32·2,

y_1 = initial water depth (ft).

All the data obtained was correlated by the following relation:

$$\frac{Q_a}{Q_w} = k(F-1)^{1\cdot4}, \tag{14}$$

where Q_a and Q_w = air entrainment rate and water flow in consistent units,

k = empirical constant.

VOL. 5 1961

There was evidence that for zero dissolved oxygen in the oncoming flow that absorption efficiencies of oxygen of above 50 per cent were obtained. On the basis of the kinetic energy in the flow approaching the jump, calculations indicate that oxygen absorption rates varying from 5 to 7 lb per hp. per hr were obtained. Thus it would appear that we have a hydraulic means for oxygenation which surpasses in efficiency anything we have discussed previously.

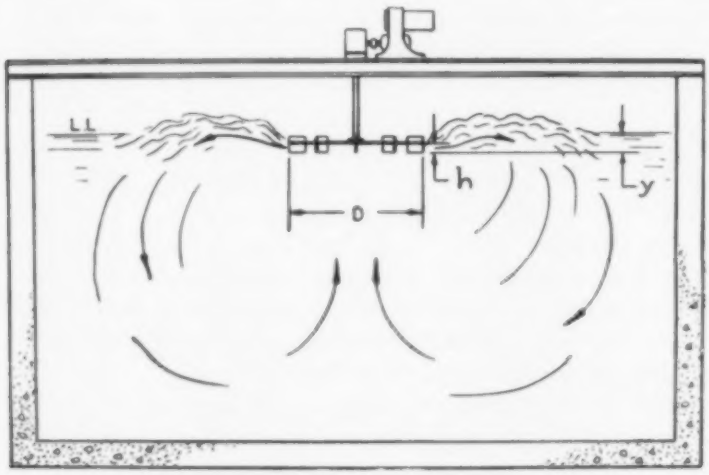


FIG. 4. A mechanical surface aerator.

The problem now was to design a mechanical device which would generate a hydraulic jump in the liquid surface with the resultant remarkable air insufflation This meant pumping a relatively large quantity of water at velocities, in accordance to Froude's law, which would produce a hydraulic jump. The simplest device of this type is a turbine so located near the liquid surface (Fig. 4), so that the desired jump is produced. It was found by experimentation that the important design factor was the Froude number written as follows:

$$F = V_p/\sqrt{V(gh)}, \tag{15}$$

where V_p = peripheral velocity of rotor (ft/sec),

g = 32·2,

h = height of rotor blades.

Studies on small models, and with later verification on full-scale units, indicated that the important design factors for such surface-located turbines could be correlated with the use of equation (14) by taking the water discharge, Q_w, from the turbine as being proportional to DhV_p. The general relation is:

$$Q_0 = KwDhV_p(F - 1)^k, \tag{16}$$

= lb of O_2 per unit time,

where K = proportionality constant, varying with h/y, see Fig. 4,

w = unit weight of liquid,

D = turbine diameter,

h = turbine blade height.

The exponent, k, was in general not too different from that in equation (14).

The turbines can be designed and positioned with respect to the surface so as to vary the water pumpage with respect to air entrainment. Thus for liquid suspensions, where appreciable mixing is needed, particularly in deeper basins, in order to keep all the solids in suspension, the amount of power needed per pound of oxygen dissolved will depend to a considerable degree on the relative amount of mixing that should be provided. However, so far as aerating activated sludge is concerned, it is quite possible to dissolve up to 6 lb of oxygen per hr per hp. supplied to the turbine under zero dissolved oxygen conditions.

The turbine operates with its top surface completely free of liquid. It should also be noted that with such surface aerators, deep basins, such as are used with compressed air, are no longer needed. The mixing action of these aerators is quite intense, the vortex below the turbine penetrating down to the basin bottom. These units pump about 5 ft³/sec per input h.p. to the turbine.

From purely power consumption considerations there is no question that such a surface aerator is one of the most efficient oxygenation device that can be devised. In order to keep power consumption to an absolute minimum the surface turbine aerator should be used for oxygenation, and if additional mixing is needed, because

of the suspended solids, additional slow-speed mixers, such as described in Section A should be added. It is not economical power-wise to increase merely the speed of the aeration turbines in order to obtain additional pumpage and mixing. Such a combination of mechanical equipment for aeration and mixing would be the most efficient installation from a power consumption standpoint; however, depending on the particular installation, the first cost might not be as favorable. Therefore, operating and first costs must be evaluated and a decision arrived at regarding what combination, if any, of surface aerators and mixers should be used to accomplish the desired result process-wise.

F. MISCELLANEOUS OXYGENATION METHODS

There have been in use at least three other schemes of oxygenation which should be mentioned, and their relative efficiency considered. Kountz (12) presents some data on these other oxygenation schemes, particularly in regard to use of packed towers and water operated injectors or aspirators.

On the basis of Kountz's data, and calculations regarding gas absorption in packed towers, considering putting in the oxygen into activated sludge mixed liquor having an initial dissolved oxygen of between 1 and 2 ppm, the oxygen input rate will not be greater than about 2 lb/hr per/h.p. and may be appreciably less.

VOL. 5 1961

Injectors and aspirators are basically inefficient oxygenating devices because of the high energy losses in such units. It does not appear that the oxygen input rate can exceed about 1 to 1·5 lb/hr per h.p.

In the mining and hydrometallurgical industry, flotation units have been used for many years which inject air into liquid suspensions by using a high-speed turbine submerged about 2 to 4 ft in the liquid mass. A comparable unit is available for sewage aeration. These turbines operate at such a speed that in the "eye" of the turbine a negative head is produced which overcomes the hydrostatic head, and if the "eye" is connected to the atmosphere, air is sucked into the liquid moving through the turbine. Basically, from a hydraulic standpoint, this is similar to air injection by aspirators. The inherent characteristic is present wherein high velocities are imparted to the liquid with a resultant inefficient use of applied energy so far as air input and liquid mixing is cncerned. Available data indicates that, depending on submergence and the mixing needed, oxygen input rates varying from 1·5 to 2·0 lb/hr per h.p. are possible.

G. SUMMARY

(1) Basic relations are presented indicating how the fundamental principles of fluid turbulence apply to design of mixing equipment.

(2) Evidence is presented which indicates that oxygenation and mixing of liquids by use of compressed air (or compressed gas for just mixing), is inherently less efficient than when such oxygenation and mixing is accomplished mechanically.

(3) The mechanical dispersion of compressed air provides flexibility and efficiency of oxygen absorption, from a power usage standpoint, not possible with submerged diffusers.

(4) Mechanical surface aerators, consisting of specially designed turbines, positioned near the liquid surface, provide high efficiency oxygenation by utilizing the inherent excellent air entrainment characteristics of the hydraulic jump.

(5) The most economical system of oxygenation and mixing, from power usage standpoint, would consist of mechanical surface aerators and slow-speed mixers, especially in large basins where the oxygen input requirements are low per unit of liquid volume.

REFERENCES

1. KALINSKE, A. A., Relation of statistical theory of turbulence to hydraulics. *Trans. Amer. Soc. Civil Engrs.* **105**, 1547 (1940).
2. KALINSKE, A. A. and PIEN C. L., Eddy diffusion. *Industr. Engrs. Chem.* **36**, 220 (1944).
3. ORLOB, G. T., Eddy diffusion in homogeneous turbulence. *J. Hydr. Div., A.S.C.E.* **85**, 75 (1959).
4. KALINSKE, A. A., Turbulence and energy dissipation. *Trans. Amer. Soc. Mech. Engrs.*, p. 41 (1941).
5. ALBERTSON, M. L. and HUNTER ROUSE, Diffusion of submerged jets. *Trans. Amer. Soc. Civil. Engrs.* **115**, 639 (1950).
6. WARD, C. N. and KESSLER, L. H., *Experimental Study of Air-Lift Pumps and Application to Design.* Univ. of Wis., Bulletin 1265 (1924).
7. KALINSKE, A. A. and KAHN, J. M., The steam-lift in a hot process water softener. Presented at Fall Meeting Amer. Soc. of Mech. Engrs., Sept. 1950, Paper No. 50-F-29.
8. COOK, M. W. and WATERS, E. D., *Operational Characteristics of Submerged Gas-Lift Circulators.* U.S. Atomic Energy Com., Paper HW-39432, Dec. 1 (1955).
9. MCCABE and ECKENFELDER, *Biological Treatment of Sewage and Industrial Wastes* vol. 1. Reinhold, New York (1956).
10. STRAUB, L. G. and ANDERSON, A. G., Experiments of self-aerated flow in open channels *J. Hydr. Div. A.S.C.E.* **84**, 1890 (1948).
11. KALINSKE, A. A. and ROBERTSON, J. M., Entrainment of air in closed conduit flow. *Trans Amer. Soc. Civil. Engrs.* **108**, 1435 (1943).
12. KOUNTZ, R. R., Total oxidation treatment. *Proc. of 11th Waste Conf., Purdue Univ.*, 1956. p. 157.

THE PERFORMANCE AND CAPABILITY OF MECHANICAL AERATORS IN ACTIVATED SLUDGE PROCESSING

T. W. LESPERANCE

Dorr-Oliver Incorporated

VOL. 5 1961

THIS paper discusses the performance and capability of turbine aerators which dissolve oxygen solely through their direct influence on sparger air. Oxygen dissolved from atmospheric air at the liquid surface is kept at a minimum. This can be accomplished by the proper application of mixing principles normally employed to produce a system which approaches perfect mixing. These aerators have the potential advantage of being able to make best use of well established theory (1, 7, 8) for the application, design, and scale-up of systems employing mixers for use in gas absorption. This can be accomplished satisfactorily if the power characteristics for the device are completely defined for all possible operating conditions and if the air input to the system can be controlled and is accurately known. One or more turbine impellers may be used to maintain the proper distribution of power between fluid shear and pumping so as to achieve the most consistent and effective results for mixing and gas absorption in an aeration system no matter what its size or shape. High oxygen transfer efficiencies are attainable with this type. Maintenance is negligible since the unit does not depend upon constriction to produce a fine bubble size.

Most conventional diffusion devices can supply oxygen at rates of from 20–80 ppm/hr at transfer efficiencies in the range of 5–15 per cent when clean. The D-O Aerator, a turbine-type aerator, can supply oxygen at any rate up to 500 ppm/hr and more. Similar machines have supplied many times this amount in some industrial aerobic fermentations. Any oxygen transfer efficiency can be selected for any design oxygen solution rate. Oxygen transfer efficiencies of 50 per cent or greater are neither unattainable nor uneconomical.

The key in achieving such results involves the determination of the optimum relationship of all physical variables involved in the make-up of the machine as a function of mechanical power input and gas rate. Important variables include: impeller size and shape in relation to its diameter, ratio of impeller diameter to tank diameter, impeller placement, sparger size and position, and submergence. An example of the problems involved in optimizing just one variable, turbine impeller spacing, is shown in Fig. 1 (4). The absence of a definite trend makes it impossible to predict the most efficient position on the basis of scattered tests. Relatively good or poor results may be shown depending upon the chance selection of test conditions. Complete success is dependent upon our ability to define the patterns produced

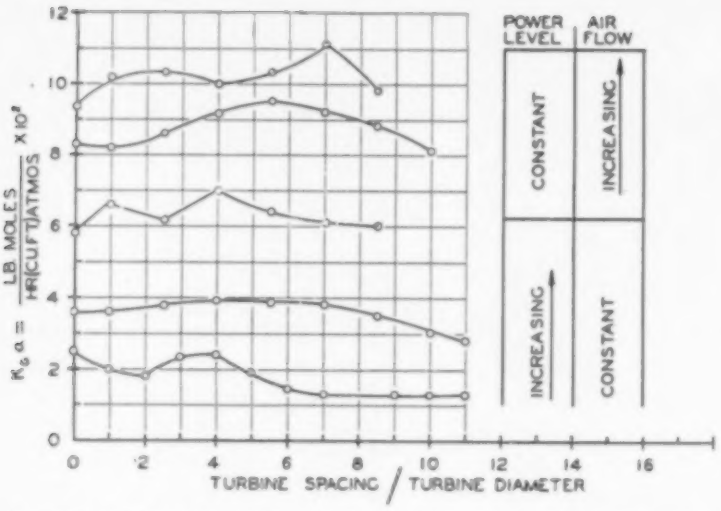


FIG. 1. Absorption coefficient vs. turbine spacing at various power levels and air flows (data from Rushton).

as a result of changing these variables so that an approach to optimum relationships can be achieved in full scale design for any specific condition.

MIXING

In a complete mixed aeration system an increment of flow introduced into the tank will be dispersed almost immediately throughout the entire tank contents. Liquor in a complete mixed system approaches the characteristics of the treated effluent rather than the raw waste and this may be of considerable importance when organic wastes are being treated which are ordinarily toxic to micro-organisms.

Complete mixing in aeration systems can be obtained by the proper choice of aeration tank configuration and by the use of mechanical aerators which permit an approach to homogeneity even in long rectangular aeration tanks fed longitudinally. Excellent front-to-back mixing can be produced by the flow patterns from a series of mechanical aerators in a rectangular tank which induces movement in both directions. The taper normally encountered in this type of system is diminished or eliminated completely. Improved distribution of the B.O.D. load in the aeration tank accomplishes an effect similarly obtained by the well known Step Aeration Process.

AERATION DESIGN CRITERIA

Once the physical features of the mechanical aerator have been determined so as to approach an optimum relationship for gas absorption and mixing, the mass transfer capability of the unit can be completely defined for any power level, aeration tank configuration, and liquid depth. The oxygen transfer coefficient for some

base or datum liquid system can be directly correlated with the mechanical power input per unit volume and superficial gas velocity. Sewage mixed liquor has been chosen as the datum system upon which to base design. Other wastes are related by comparative transfer rates.

Test data, to date, indicate that overall transfer coefficients generally vary plus or minus 25 per cent, from sewage mixed liquor—the datum. Oxygen solubility rarely exceeds and may be as much as one-half that for water or sewage mixed liquor. This is important since it sets the maximum potential concentration driving force available for transfer.

When the necessary process information has been provided the problem of aerator selection begins. Almost an infinite number of power splits can be used to supply a single design oxygen uptake rate. By power split, we mean the distribution of the total, between mechanical power input and that required for blowing air which, in turn, determines the oxygen transfer efficiency. There is an optimum power split which will yield the lowest total power consumed. This may not necessarily be the choice for design since other considerations such as flexibility and foaming problems may dictate a different power split.

Fig. 2 shows variation in oxygen transfer efficiency with liquid depth at a constant mechanical power input and superficial gas velocity. For all practical purposes,

VOL. 5 1961

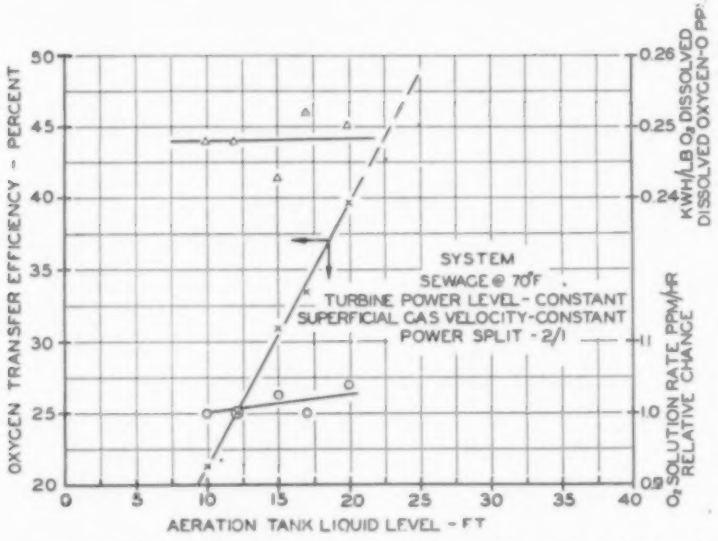


FIG. 2. Oxygen transfer efficiency, capacity, and power variation with design aeration tank and liquid level.

the performance of the D-O Aerator is independent of liquid depth with respect to oxygen solution rate and power consumed per pound of oxygen dissolved. Oxygen transfer efficiency and liquid depth can be important design considerations where

foaming problems are encountered in activated sludge aeration tanks. The D-O Aerator can be installed in tanks with liquid depths up to 40 ft.

Dissolved oxygen level may vary the oxygen transferred at any total power input by as much as plus or minus 30 per cent over the range of 0·5–2·0 ppm usually employed. This should be taken into consideration when analyzing such design problems as power requirement, safety factor, and flexibility.

D-O AERATOR PERFORMANCE

Table 1 presents representative full-scale and pilot plant operating data. The high oxygen solution capability of the D-O Aerator has permitted the treatment of

TABLE 1. D-O AERATOR—ACTIVATED SLUDGE PLANT OPERATING DATA

Waste	Raw waste B.O.D. ppm	Aeration tank (°F) temp.	O_2 consumption (ppm/hr)	B.O.D. removal (lb/1000 ft³)	B.O.D. removal (%)	Oxygen transfer (kWh/lb·O_2) (Absorbed)	Oxygen transfer (%)
				655	94	0·31	20
Paper mill	2250	107	370	655	94	0·31	20
Sewage	157	72	62	82	90	0·24	25
Pulp mill, Kraft	845	96	206	300	86	0·34	21
Chemical	2870	97	82	75	95	0·3	35
Packing house	870	80	416	358	92	0·11	26
Pulp mill, sulphite	3100	97	588	845	89	0·21	28

Note: All data obtained at steady-state operating conditions.
Power data adjusted to 0 ppm dissolved oxygen.
Power for air based on blower b.h.p., 5 per cent.

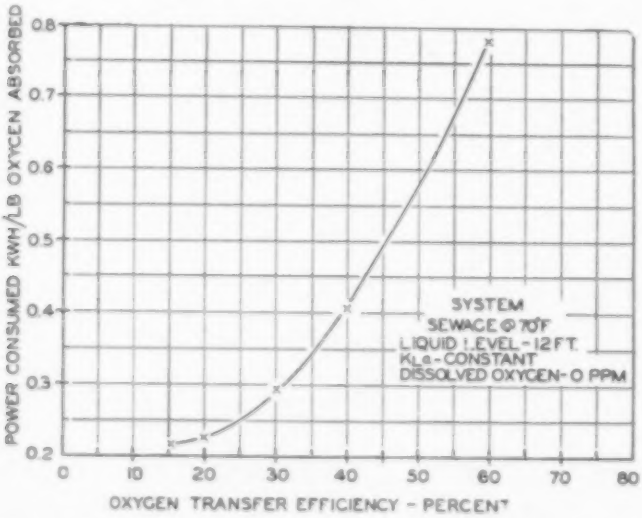


FIG. 3. Power requirement as a function of oxygen transfer efficiency.

large amounts of B.O.D. in relatively condensed systems. Of course, other favorable process conditions such as high temperature and B.O.D. concentration must be present to permit the treatment at the highest B.O.D. loadings per unit aeration volume. These conditions, however, are not uncommon in many industrial wastes.

Maintenance is negligible on this equipment apart from routine lubrication. Since the sparger ring holes vary in size from $\frac{1}{2}$ to $1\frac{1}{4}$ in., clogging is not a problem. Flexibility can be obtained by the use of additional blower capacity and of change gears to increase turbine speed. Since low air flows are frequently used, foaming is less than that experienced from diffused air operation. Fig. 3 illustrates the variation in power consumed as a function of oxygen transfer efficiency at one liquid level. No deleterious effect on sludge quality due to fluid shear or turbulence has been observed up to 26 ft/sec peripheral speed.

Fig. 4 shows power consumed per pound of oxygen dissolved as a function of oxygenation capacity or rate. This is of interest since we are considering and recom-

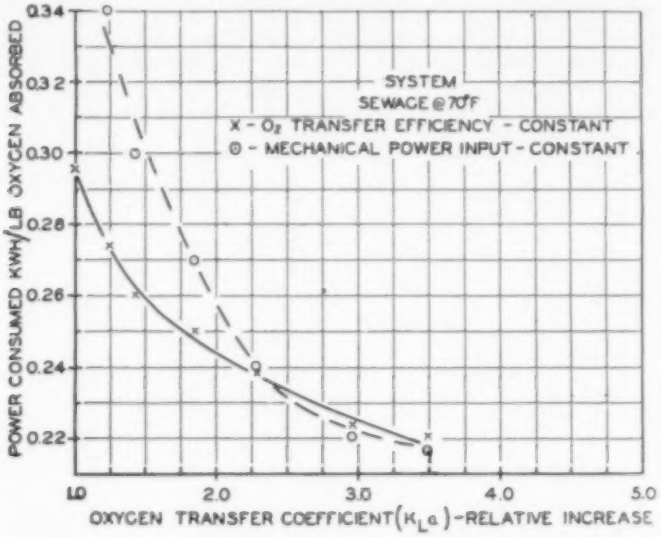


FIG. 4. Power required as a function of oxygenation capacity.

mending application of mechanical aerators under design conditions where the oxygen utilization rate will be very high. It is interesting to note that power consumed per pound of oxygen dissolved decreases with increasing mixing and aeration intensity. Operation of activated sludge plants at power levels higher than we now normally use may result in lower overall power consumption.

SUMMARY

Mechanical aeration equipment available today gives the engineer several additional degrees of freedom in the design of activated sludge treatment facilities. Plant size and shape can be based solely on the requirements dictated by the needs of the

VOL. 5 1961

biochemical process, site limitations, and economics of plant construction. Mechanical aerators can exhibit extreme versatility with respect to oxygen supply. They are economic in both plant cost and power consumption, low in maintenance, flexible, and demonstrate desirable characteristics for foam control or elimination.

REFERENCES

1. OLDSHUE, J. Y., Aeration of biological systems using mixing impellers. *Biological Treatment of Sewage and Industrial Wastes* pp. 231–240. Reinhold, New York (1956).
2. IPPEN, A. T. and CARVER, C. E., Basic factors of oxygen transfer in aeration systems. *Sewage Industr. Wastes* **26**, 813–29 (1954).
3. MCKINNEY, R. E., Complete mixing of activated sludge. *Wat. Sewage Wks.* **107**, 69–73 (1960).
4. RUSHTON, J. H., GALLAGER, J. B. and OLDSHUE, J. Y., Gas–liquid contacting with multiple mixing turbines. *Chem. Engng: Progr.* **52**, 319–23 (1956).
5. RUSHTON, J. H., The use of pilot plant mixing data. *Chem. Engng. Progr.* **47**, 485 (1951).
6. RUSHTON, J. H., Power characteristics of mixing impellers. *Chem. Engng. Progr.* **46**, 395–404 (1950); and 467–76 (1950).
7. OLDSHUE, J. Y., Application of mixers to bio-engineering processes. *Proc. Bio-Engineering Symposium, Rose Polytechnic Institute, Terre Haute, Indiana, May 1953.*
8. OLDSHUE, J. Y., Theory and design of mixers for aeration of waste. *Proc. 10th Ind. Wastes Conf-Purdue Univ., May 1955.*
9. RUSHTON, J. H., *Technology of Mixing*. Canadian Chemistry and Process Industries (1946).

POWER REQUIREMENTS FOR OXYGEN TRANSFER WITH TURBINE AERATORS

C. BUELTMAN, G. WESSON, J. TERMINI and W. KINGSBURY

Pfaudler-Permutit, Inc.

THIS paper reports the results of pilot plant experiments with a dual impeller turbine aeration unit. The experimental studies were conducted in a tank 7 ft in diameter with a 12 ft liquid depth. The tank was equipped with a 10 h.p. motor, with a variable speed reducer. Four equally spaced baffles were attached to the side of the tank to simulate rectangular tank conditions.

Several turbine designs were evaluated. Data for purposes of this paper will be based on standard six-blade turbines with solid blades at 90° to the turbine. The turbine diameter to blade width to blade height was approximately 5/1·25/1. Turbine diameters of 10 in. 14 in. and 21 in. were checked.

The air sparger ring consisted of a 3 in. pipe segment inside a 6 in. pipe segment, with top and bottom made from flat plate. Approximately twenty-nine equally spaced 42 drill holes were located around a $4\frac{7}{8}$ in. diameter circle.

VOL. 5 1961

Approximately 10 ft of water was used for the tests. The bottom turbine was centrally located about 21 in. from the bottom of the tank. The air sparger was centered just below the lower impeller. The upper turbine was placed at various submergences below the water surface.

Four aerator designs were compared:

Type I consisted of a bottom impeller located at 21 in. from the bottom of the tank for air dispersion.

Type II consisted of an additional turbine located at approximately $49\frac{1}{2}$ in. below the water surface for mixing purposes.

Type III consisted of a top turbine for surface aeration with a bottom turbine for mixing (no sparger air flow).

Type IV (Permaerator) was a combination of the bottom impeller (with sparger air flow) plus a top turbine (for surface aeration) submerged at from 2·5 in. to 18 in. from the water surface.

DISCUSSIONS OF RESULTS

Submergence of top impeller

Dual turbine aeration with the top impeller located close to the water level results in enhanced oxygen absorption due to surface aeration and the additional dispersion of air bubbles injected through the sparger. This action is illustrated in Fig. 1.

The quantity of oxygen transferred from surface air is dependent on the submergence ratio (diameter of top impeller to submergence depth), impeller tip speed, impeller diameter and air flow.

Fig. 2 shows the relationship between submergence ratio to oxygen transfer rate and power consumption for the conditions of no air flow and for a tip speed of approximately 750 ft per min. An optimum submergence exists for a specific

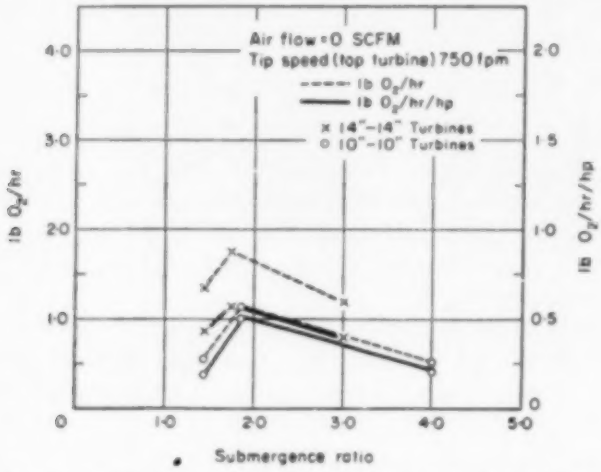


FIG. 2. Dual turbine aeration: Effect of submergence of top turbine on transfer rate and power for no air flow, at top impeller tip speed of 750 ft per min, and with bottom turbine located 21 in., from bottom of tank.

turbine diameter and operating conditions. Oxygen transfer and power economy decreases, if the top impeller is either too deep (decreased renewal of surface air in contact with liquid) or too shallow (insufficient water is agitated and contacted with air).

Air rate

Air is supplied to the lower turbine through a sparge ring, and is dispersed into small bubbles by the rotating action of the impeller blades. For a fixed turbine diameter rotating at a constant speed, the oxygen absorption rate is a function of gas flow. Increased oxygen transfer rate is usually obtained with smaller bubbles (larger surface area for transfer) and with increased air rate (greater interfacial area). However, if the sparger air flow becomes too high, the impeller becomes surrounded with a stream of gas that is not effectively dispersed, and liquid is displaced from the turbine by the air path, resulting in inefficient gas-liquid contact. This condition is called "flooding". The net result is a decrease in the oxygen transfer rate, even though the air volume increases.

Fig. 3 shows this phenomenon for a constant peripheral speed of the top impeller. A 10 in. top–10 in. lower turbine combination shows no flooding for the air flows used, since the tip speed of the lower turbine is sufficient for thorough air dispersion. A 21 in.–10 in. combination, based on operating at a constant tip speed of the top impeller, results in a lower shaft speed. Consequently, the tip speed of the 10 in.

VOL.
5
1961

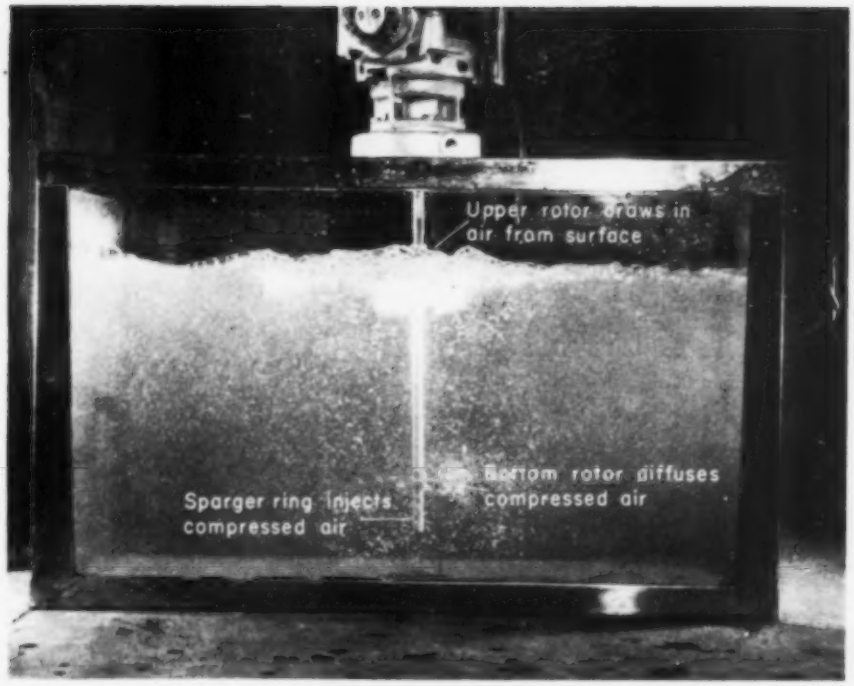


FIG. 1. Laboratory Type IV (permaerator), illustrating flow patterns and bubble dispersion.

lower turbine decreases and is not sufficient for proper air dispersion. Increasing the size of the lower turbine to 14 in. improves the aeration performance and flooding does not occur below approximately 20 scfm.

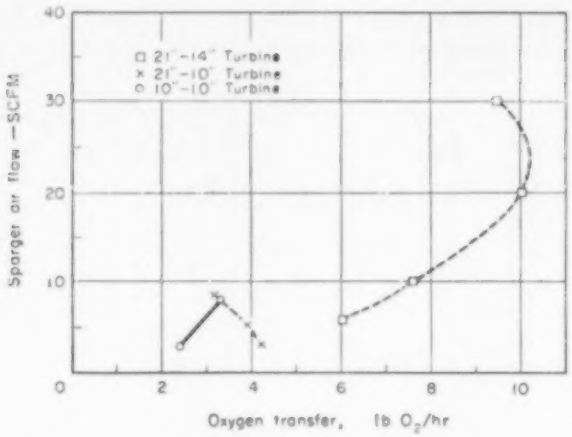


FIG. 3. Effect of air flow rate of oxygen transfer using dual turbine aeration with various combinations of turbine sizes.

VOL. 5 1961

Turbine speed

Increasing turbine tip speeds increases oxygen transfer rate. This is shown in Fig. 4 for the range of from 400 to 800 ft per min., when using a Type IV design, (14–14 in. turbine combination, submergence ratio of the top impeller at 1·75, air flow of 5–20 scfm). However, increasing turbine tip speed has an economic limit, since the turbine horse power increases. Therefore, an optimum tip speed exists for efficient power consumption. Optimum tip speed is dependent upon operating conditions, including sparger air rate as illustrated in Fig. 4.

Approximately 1·5 to 3·0 lb oxygen per hr per horse power (total, blower and turbine) was obtained for the specific conditions shown. Higher ranges of transfer per horse power (blower and turbine) are obtainable with different turbine and operating combinations.

Aerator design

Dual turbine aeration (Types II and IV) was compared with a single turbine design (Type I) at various air rates and submergence of the top impeller but for a constant speed. One set of test conditions consisted of a 14 in.–14 in. dual turbine combination with the top turbine located at 8 in. (Type IV) and 49·5 in. (Type II) from the water surface. In every case, the lower turbine was 21 in. from the bottom of the tank.

Fig. 5 shows that dual turbine aeration, particularly when the top impeller is located a critical distance below the water surface, results in appreciably higher

oxygen transfer rates than that obtained with the single turbine. Consideration of power requirements changes the relative picture.

Dual turbine aeration requires greater turbine power than the single turbine design. However, in each instance the Type IV design resulted in a greater increase in oxygen transfer than the increase in turbine horse power. Type IV therefore provides the most economical design, combining the highest transfer rate with the lowest horse power increase.

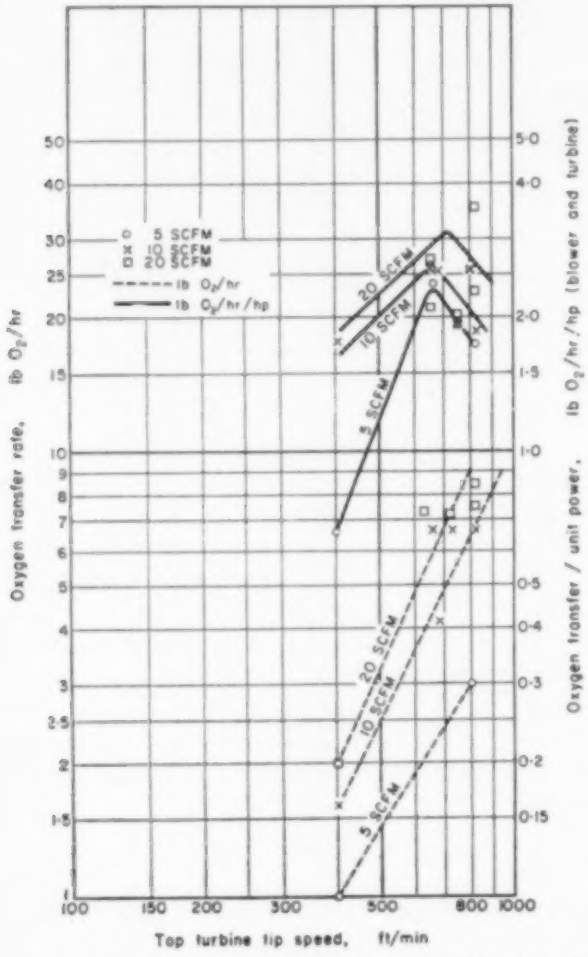


FIG. 4. Effect of impeller speed on oxygen transfer and power for 14 in.–14 in. dual turbine system Top turbine submergence ratio equals 1·75, and bottom turbine is 21 in. from bottom of tank.

Type I and II designs have been found to require relatively more power for increased transfer. Under certain conditions it was found that Type II resulted in less oxygen transfer per unit power than Type I, but in general Types I and II were found to be in the same range.

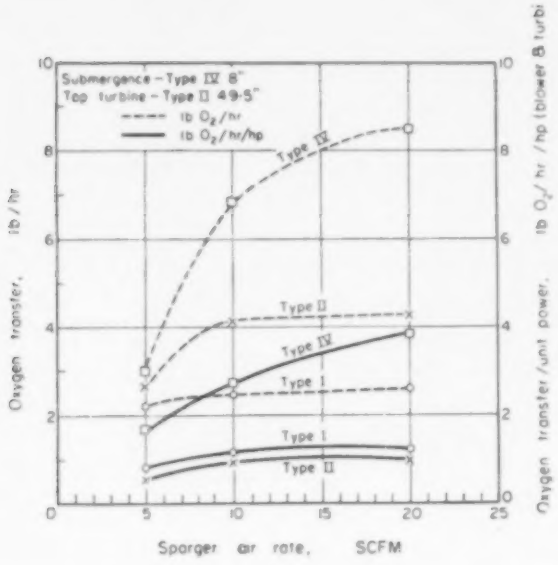


FIG. 5. Dual turbine, 14 in.–14 in. (Types II and IV); compared to single turbine, 14 in. (Type I); At constant tip speed of 804 ft per min. Lower turbine located 21 in. from bottom of tank.

VOL. 5 1961

CONCLUSIONS

Based on the data obtained for the test conditions used, the following conclusions are made:

(1) For dual turbine aeration, there exists an optimum submergence level for the top turbine. This level will be controlled by the sparger air flow rate, turbine tip speed and turbine sizes.

(2) If the sparger air rate becomes too high, flooding occurs and the oxygen transfer rate actually decreases. The flooding velocity is dependent principally on the size and speed of the lower turbine.

(3) Increasing the impeller speed will increase the oxygen transfer rate. However, excessive speeds are not desirable because of the unattractive increase in power requirements.

(4) Dual turbine design of the Permaerator type shows significantly better oxygen transfer rates and power consumptions than a single turbine or a dual turbine with the top turbine submerged below the optimum submergence level.

DIFFUSED AIR OXYGEN TRANSFER EFFICIENCIES

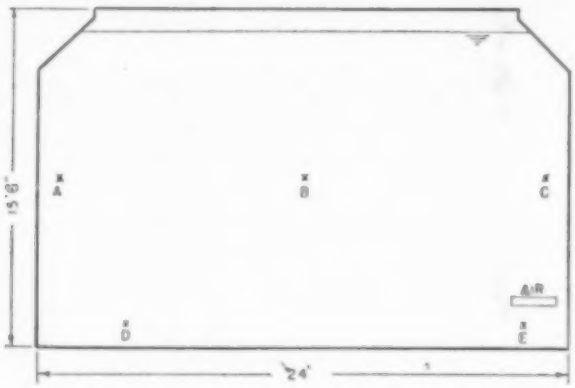
P. F. MORGAN and J. K. BEWTRA

State University of Iowa

A TEST tank 4 ft long, 24 ft wide and 15·5 ft total depth was employed for these studies. Oxygen transfer studies were conducted on cobalt-catalyzed sodium sulfite deaerated water. The details of the test procedure have been previously reported (1).

All samples were taken at sampling point A as shown in Fig. 1. However, as a check a series of simultaneous samples were taken at the five sampling points shown for an air flow of 16·70 ft³/min. The results, expressed in per cent oxygen absorption, are shown in the table in Fig. 1. All samples were within 2·75 per cent of the mean and the samples from A were the same as the mean. This indicates that samples from point A truly represent the complete tank contents for the diffuser arrangements shown in Fig. 1 and that the deviation resulting from sampling at various points in the tank is well within the 5 per cent accuracy desired for this work.

VOL.
5
1961



AIR FLOW = 16.70 SCFM

OXYGEN ABSORPTION, MEAN OF 5 SAMPLING POINTS = 10.49 %

SAMPLING POINT	OXYGEN ABSORPTION %	DEVIATION FROM MEAN %
A	10.5	0
B	10.5	0
C	10.3	-1.65
D	10.75	+2.75
E	10.4	-1.05

FIG. 1. Uniformity of oxygen absorption in tank.

SARAN BUBBLE DIFFUSERS

Standard saran diffuser tubes 2 ft long by 3 in. outside diameter wound 20 turns per in. were mounted 2 ft above the bottom of the tank, and adjacent and normal to the 4 ft side of the tank. This setting provided a submergence of 12 ft 9 in. Oxygen absorption efficiencies were determined with two tubes (one tube per each 2 ft length of tank) with four tubes and with six tubes. The resulting efficiency curves are shown in Fig. 2. At the rated capacity of 6 ft³/min per tube, the efficiency varies

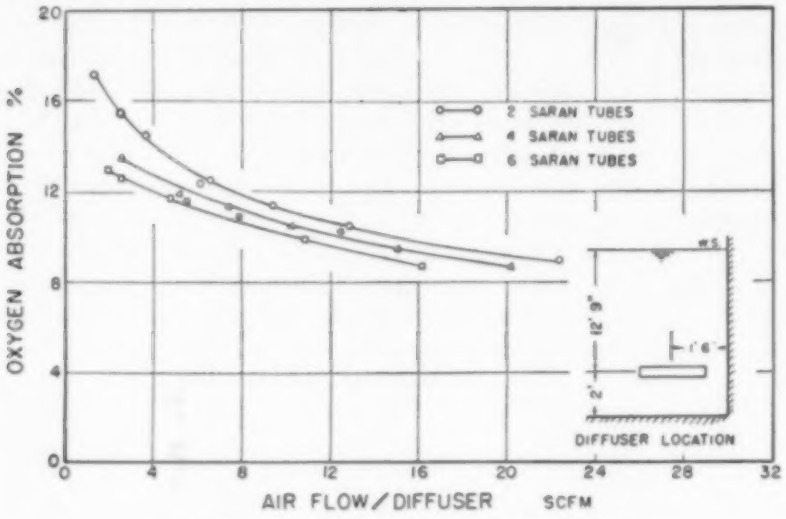


FIG. 2. Saran tube efficiency—air flow per diffuser.

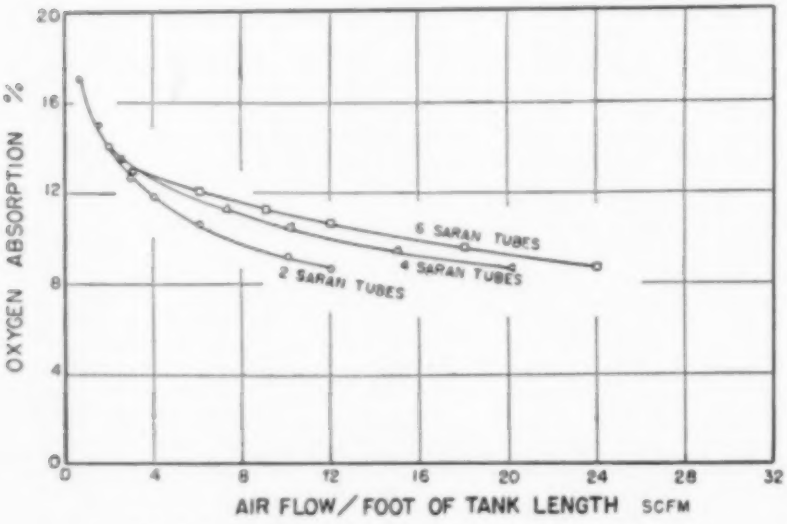


FIG. 3. Saran tube efficiency—air flow per foot of tank.

from 11·3 per cent with 1½ tubes* per ft of tank to 12·6 per cent with one tube per 2 ft length of tank. The same data has been replotted in Fig. 3 to show oxygen absorption efficiency for various air flows per foot of aeration tank with 2, 4 and 6 tubes in the 4 ft length of tank. For the diffuser arrangements shown on the drawings, it is apparent that for any given air demand per ft length of tank, increasing the number of tubes or reducing the rate of flow per tube results in improved efficiency.

SPARGER TYPE DIFFUSERS

Spargers, each with four 5/16 in. diameter holes radiating horizontally from the center were checked for oxygen absorption efficiency and results are shown in Fig. 4. The efficiency is shown for 2, 4, and 6 spargers per 4 ft length of tank. In

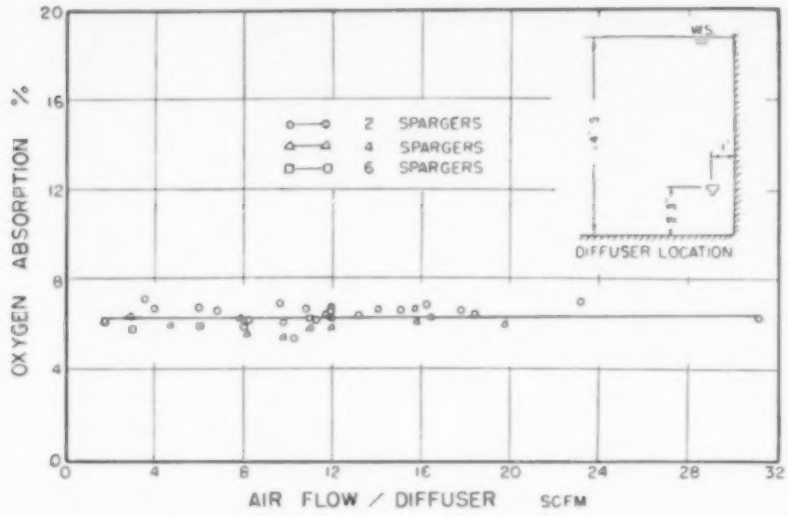


FIG. 4. Sparger efficiency—air flow per diffuser.

each of the tests the diffusers were mounted in the conventional position, in a row 1 ft away from the wall and at an elevation such that the air discharged 2 ft 3 in. above the tank floor to provide 12 ft 2 in. of submergence.

The slightly higher efficiency points at the low flows per foot of tank with two diffusers, may be interpreted as indicating a slightly higher efficiency at low circulating velocities or may be accepted as normal variation of test results.

THE EFFECT OF DIFFUSER SUBMERGENCE AND WATER DEPTH

The e fect of varying diffuser submergence was studied in two ways. For one series, the water depth in the tank was maintained constant at full depth and the dif-

* All results are expressed in per cent oxygen absorption in tap water at 20°C and zero dissolved oxygen.

VOL. 5 1961

fusers were moved in increments from the bottom of the tank to within 3 ft of the surface. For the other series the diffusers were held in the conventional spiral flow position and the water depth in the tank was varied.

Two saran tubes (one tube/2 ft of tank) were operated at various elevations from the tank bottom to within 3 ft of the surface of the tank. The results are shown in Fig. 5 for air flow rates of 6 and 12 ft³/min per tube.

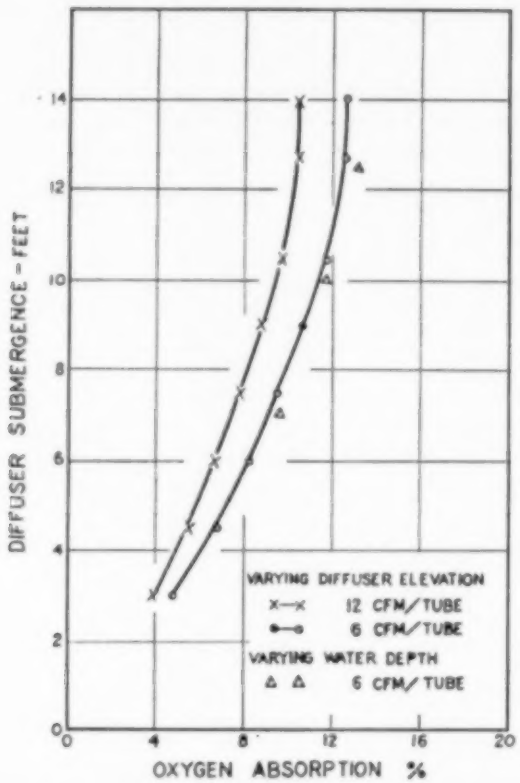


FIG. 5. Effect of diffuser submergence on oxygen absorption—two saran tubes.

Then the two tubes were held in the conventional spiral flow position, 2 ft above the tank bottom, the tank coping was removed and the tubes were operated with 9 ft, 12 ft and 14 ft 5 in. water depth (7 ft, 10 ft and 12 ft 5 in. submergences). These results are also shown in Fig. 5. The only difference in operating conditions at the 6 ft³/min per tube and 12 ft 5 in. submergence is that the "varying tube elevation" point was with coping and the "varying water depth" point was without coping. It would therefore appear that the conventional coping on aeration tanks reduces the efficiency from 13·2 to 12·6 per cent. However, this is not a significant difference since the variation is within the 5 per cent of accuracy limit of this work. It is, however, significant that all three points for "varying water depth" differ

from the "varying tube elevation" curve by this same amount and if these points were adjusted for the apparent effect of coping, the results would coincide exactly for given submergences. It can be concluded that the absorption efficiency with one saran tube per 2 ft length of tank is a function of the diffuser submergence and that it is independent of the water depth in the tank within the range of this study. It can also be concluded that the coping does not have a significant effect on the rate of oxygen absorption within the range of these tests. It also appears that lowering the diffusers below the conventional spiral flow position (2 ft above tank bottom) would not increase the absorption efficiency.

A similar study was made for spargers. Two spargers, similar to those used previously, were operated at various elevations from 2 ft 3 in. above the tank bottom to within 3 ft of the surface in 14 ft 5 in. water depth tank. The results are shown in Fig. 6 and it is apparent that the percentage oxygen absorbed is directly proportional to the diffuser submergence.

A second series of samples were taken with the spargers in the conventional spiral flow position, 1 ft from the wall and 2 ft 3 in. above the tank floor, and the water depth in the tank was varied from 14 ft 5 in. to 6 ft 9 in. These results are also shown

VOL. 5 1961

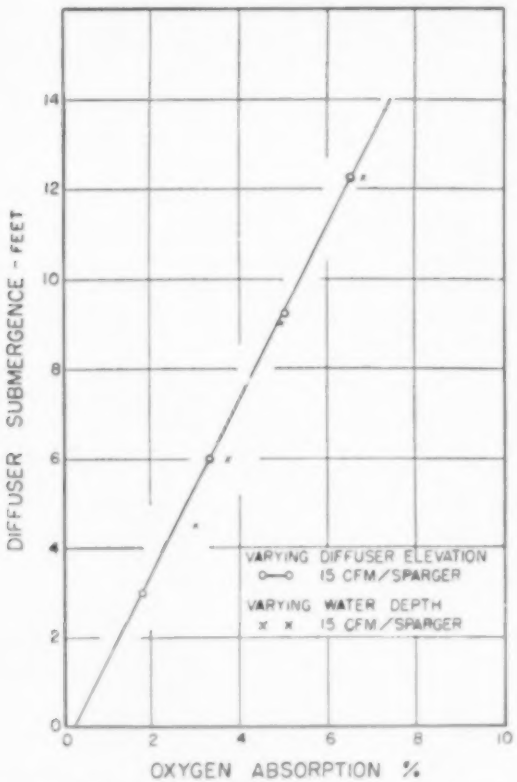


FIG. 6. Effect of diffuser submergence on oxygen absorption—two spargers.

as "varying water depth" points on Fig. 6. At 12 ft 2 in. submergence the only difference in the two points is that the tank coping has been removed for the "varying water depth" curve. This would indicate that oxygen absorption is increased from 6·5 per cent to 6·8 per cent by removing the coping. All points on the "varying water depth" curve are this same amount greater than the corresponding submergence on the "varying tube elevation" curve.

In other words except for the effect of the coping, the absorption is the same for any given tube submergence regardless of whether the submergence is obtained by changing the water depth in the tank or by changing the tube elevation.

It is also of interest to note that these curves on effect of submergence indicate that, in a spiral flow tank, the amount of oxygen absorbed at the time of bubble release at the tank surface is negligible because as the submergence approaches zero, the oxygen absorption with spargers approaches zero.

Acknowledgement—These studies were carried on in the Domestic and Industrial Wastes Laboratory at the State University of Iowa, Iowa City, Iowa, and were financed by a University research grant from the Chicago Pump Company, Chicago, Illinois.

The authors are particularly indebted to Mr. William R. Nicholas and Mr. Dilman Stachour, Research Assistants and to Mr. Allen Poole for their help in conducting these tests.

REFERENCE

1. Morgan, P. F. and Bewtra, J. K., Air diffuser efficiencies. *Water Pollution Control J.* **32**, 1047–59 (1960).

DISCUSSION

Douglas E. Dreier: We have utilized oxidation of sodium sulfite, always catalyzed with 1 ppm of cobalt, as a basis of oxygen transfer studies in our 6500 gal test tank (12 ft × 5 ft × 14 ft 6 in. W.D.) In our large tank we have found that the rate of reaction is a function of the catalyst concentration, to a minor degree, up to about 1 ppm of cobalt. However, the variation with catalyst concentration in our large tank is much less than that shown for the small laboratory tank at the University of Iowa. By always using the same catalyst concentration we have obtained good duplication of results. Also, this amount of catalyst seems to give results quite comparable to field measurements in activated sludge tanks.

Table 1 shows typical absorption efficiencies at optimum air flow rates for various air diffusion devices in various applications. The latter five results were from data obtained by Eckenfelder. At one pulp and paper activated sludge plant, a statistical plot of almost a month's data indicated an average transfer efficiency of 9·8 for Walker spargers in a tank 24·5 ft wide × 15 ft W.D.

Table 2 contains data taken under Eckenfelder's direction by polarographic study of dissolved oxygen and oxygen utilization rates in two parallel activated sludge tanks (each 350 ft × 30 ft × 15 ft W.D.) at San Antonio, Texas. There were 512 saran tubes in one tank and 264 spargers in the other. The tubes were in a tapered pattern and the spargers were uniformly spaced.

Tests taken at about this same time at Bergen County, N. J., where spargers replaced saran tubes due to tube clogging difficulties, indicated an average of 9·6 per cent in the sludge stabilization tank (14·8–15·5 ft^3/min per sparger), and 7·5 per cent in the contact zone with a high air rate range of 22–28 ft^3/min per sparger. No direct comparison could be made in

this plant, since all tubes in use were replaced, but it is of interest that after 3 years of experience the plant is presently being enlarged with spargers.

Table 3 shows a typical comparison of new saran tubes and spargers in oxidizing sulfite in our 6500 gal test tank.

TABLE 1. OXYGEN ABSORPTION EFFICIENCY OF AERATION DEVICES

Diffuser unit	Waste	Depth (ft)	Air flows (scfm/unit)	Oxygen[1] absorption (%)
Aloxite Tubes	Sulfite	15	6	13·7
Plastic Tubes	Sulfite.	15	6	10·0
Sparger	Sulfite	15	16	10·0
Impingement	Sulfite	15	12	11·7
Disc-type	Sulfite	13	16	9·8
Sparger[2]	Sewage	15	16	8·6
Sparger	Pulp and paper	15	16	9·0
Sparger[3]	Pulp and paper	15	16	9·8
Disc-type[3]	Pulp and paper	15	16	8·6

[1] At zero dissolved oxygen.
[2] Activated sludge plant.

VOL. 5 1961

TABLE 2. OXYGEN ABSORPTION EFFICIENCY (PER CENT)[1]

	Jan. 31, 1957	Feb. 1	Feb. 2
Spargers	8·0 (15·0)	10·40 (10·0)	10·45 (12·4)
Saran tubes	6·75 (7·5)	8·95 (5·25)	10·45 (6·35)

[1] Cubic feet per minute of air used per diffuser shown in parentheses following per cent absorption.

TABLE 3. SULFITE OXIDATION

Diffuser	No. of units	Air flow (scfm/unit)	Depth (ft)	Oxygen absorbed ppm/hr	K_{LA}	Oxygen absorbed (%)
Sparger	3	4	13	14·2	1·32	6·2
,,	3	8	13	34·3	3·20	7·62
,,	3	12	13	64	5·92	9·60
,,	3	16	13	89·5	8·30	9·87
New saran tubes	3	4	13	20·5	1·9	9·23
,,	3	6	13	34·8	3·2	10·40
,,	3	8	13	45	4·17	10·10
,,	6*	6	13	69·6	6·45	10·40

* In wide band pattern of two rows of three tubes each.

The oxygen absorption rates are typical of values found in activated sludge and do not indicate that sulfite is valid only to simulate biological suspensions with very high oxygen utilization rates.

Neither aeration of deoxygenated water nor of sodium sulfite gives direct information about rates of transport of oxygen from solution to the respiratory enzymes in the sludge floc. It is of interest that our sulfite oxidation results for saran tubes (except at 4 ft³/min) fall quite close to Morgan's curves for deoxygenated water; however, his measurements show sparger efficiency as only 50–60 per cent of that for tubes.

We have not used deoxygenated water as a standard test since we found that the direct measurement of a process result in sulfite oxidation is a much simpler and less sensitive technique and much less subject to error at the high air release rates. When utilizing the two-film theory and calculating the log slope, it is necessary to assume the saturation value which controls the transfer coefficient. For porous diffusers which release bubbles with essentially an isolated rise pattern once formed, and with very limited surface aeration, the use of mid-depth may be a reasonably valid assumption for the C_i value.

For a device such as the Walker sparger, which obtains a considerable portion of its transfer from extremely turbulent surface aeration and from fine air carried throughout the tank by the higher circulating velocities established, the correct value of C_i cannot be computed nor readily determined*. Also, with the spargers, there is a continuous re-forming of smaller bubbles from that portion of the air which momentarily coalesces in the concentrated, turbulent air rise pattern, and much of the air is worked into the higher initial transfer rate. Since the derivation of the two-film theory assumes that C_i remains constant, and that there is no appreciable change in the surface contact area of the bubbles,† there is considerable question as to whether this procedure is properly applied to spargers. The overall transfer coefficient K as used in the deoxygenated water technique is based on bubble area alone, and because of the influence of surface aeration the curve does not plot as a straight line excepting at relatively low dissolved oxygen values, and K gradually decreases with time. This was pointed out by Dobbins,‡ and has been verified in our studies.

TABLE 4. DEOXYGENATED WATER DATA

Diffuser	No. of units	Air flow (scfm/unit)	Depth (ft)	Oxygen absorbed (%)
Sparger	3	8	13	7·85
	3	12	13	9·0
	3	14	13	9·6
	3	16	13	10·1
Saran tubes (new)	3	4	13	11·9
	3	6	13	11·0
	3	8	13	10·1

Table 4 shows the average values of our studies to date with deoxygenated water in our large test tank, based on C_i assumed at mid-depth for both tubes and spargers. The sparger values would increase for C_i values less than mid-depth.

* ECKENFELDER, W. W., JR., Factors affecting the aeration efficiency of sewage and industrial wastes. *Sewage Wks. J.* **31**, 1, 60 (1959).

† IPPEN, A. T. and CARVER, C. E., JR., Basic factors of oxygen transfer in aeration systems. *Sewage Wks. J.* **26**, 7, 813 (J. 1954).

‡ DOBBINS, W. E., Discussion. *Sewage Wks. J.* **26**, 7, 827 (1954).

If deoxygenated water is to be used as a general check on sparger efficiency, then some value of C_t should be assumed at a point less than mid-depth to attempt some allowance for surface aeration and other effects peculiar to the sparger.

We do not concur that the deoxygenated water technique as applied to spargers can be considered a "standard" test for oxygenation efficiency due to the factors discussed. We believe that parallel field tests in activated sludge tanks is the final test of performance, coupled with the long range relative maintenance costs.

AUTHORS: The sulfite method of measuring oxygen absorption was studied in the full scale tank in addition to the small laboratory tank before it was finally abandoned. Figs. 7 and 8 show the comparison of results obtained with the two methods in the full scale tank. The sulfite used for this study was "Nalco" no. 19, a sodium sulfite with cobalt catalyst which was supplied by the Nalco Chemical Company. This product provides a uniform distribution of the cobalt chloride in the sulfite salt and it has a reaction rate equivalent to sodium sulfite with approximately 0·5 ppm cobalt chloride as Co^{2+}.

The efficiency by this method is expressed as equivalent rate of oxygen absorption at 500 ppm concentration of sulfite as SO_3^{2-}. It is apparent, at least with the diffuser tubes, that there is no fixed relation between the sulfite and the deoxygenated water method.

The general use of the deoxygenated water method as indicated by a survey of current literature indicates the general acceptance of this method by sanitary engineers in preference to the sulfite method.

In the studies at the University of Iowa all commercial diffusers tested have been standard units available for sale by the company supporting the research contract. No competitors' aeration devices have been tested at any time.

The comment is made that a sparger "obtains a considerable portion of its transfer from the extremely turbulent surface aeration and from the fine air carried throughout the tank by the higher circulation velocities established (so) the correct value of C_t cannot be computed nor readily determined".

VOL. 5 1961

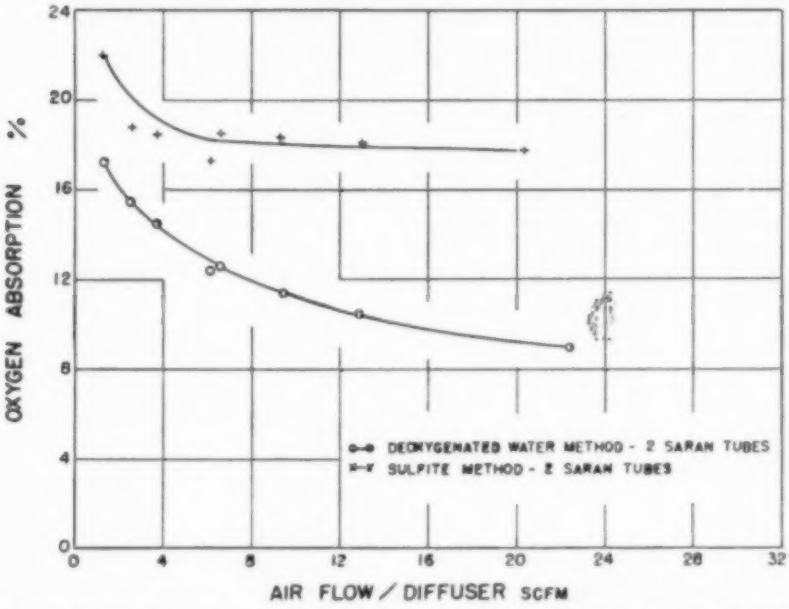


FIG. 7. Comparison of deoxygenated water method with sulfite method—saran tubes.

The curve in Fig. 6 indicates, by the intercept of its abscissa, that the absorption due strictly to surface aeration must be negligible. This does not mean the circulation is not effective in the overall oxygen absorption process but it cannot be responsible for a "considerable portion" of the transfer.

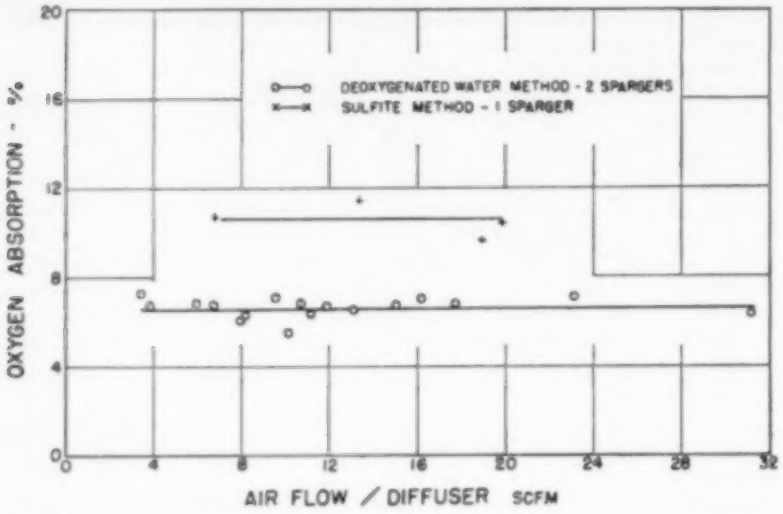


FIG. 8. Comparison of deoxygenated water method with sulfite method—spargers.

The selection of the saturation value (C_i) for mid depth pressure was adopted only after a careful study of this problem. If the formula for the oxygen absorption rate

$$\frac{dC}{dt} = \frac{10^6}{W} KA(C_i - C_L),$$

is transposed to the form

$$\log_{10} \frac{C_i - C_0}{C_i - C_L} = \frac{10^6}{2 \cdot 30 W} KAt$$

and if the axis is adjusted so that

$$C_0 = \text{zero at } t = \text{zero},$$

then

$$\log_e \frac{C_i}{C_i - C_L} = \frac{10^6}{W} KAt;$$

let

$$\frac{10^6}{W} KA = K',$$

then

$$\log_e \frac{C_i}{C_i - C_L} = K't,$$

$$\frac{C_i}{C_i - C_L} = e^{K't}, \quad \text{and}$$

$$C_L = C_i(1 - e^{-K't}).$$

This is similar to the form of the B.O.D. equation. The unknown values of K' and C_i were then determined by both the Thomas slope method and the moment method. Table 5 shows the values for K' and C_i for six typical runs. Table 6 shows the oxygen absorption computed with C_i as determined by the Thomas slope, the moment method and the

TABLE 5. VALUES OF C_i AND K' COMPUTED BY DIFFERENT METHODS

Run No.	Diffusers	Log method		Slope method		Moment method	
		C_i	K'	C_i	K'	C_i	K'
1	2 saran tubes	11·20	0.0288	12·08	0·0262	17·20	0.0175
2	,,	11·20	0·0471	11·25	0·0458	13·07	0·0379
3	,,	11·20	0·0694	11·21	0·0687	10·75	0·0730
	Average			11·51		13·67	
4	2 spargers	11·20	0·0248	11·30	0·0224	16·70	0·0150
5	,,	11·20	0·0407	9·83	0·0486	10·48	0·0460
6	,,	11·20	0·0550	10·40	0·0623	9·56	0·0700
	Average			10·51		12·25	

"standard" log method. With the log method C_i = saturation at mid depth. This table also shows that all average results are well within 5 per cent of each other. The deviation of the spargers is not greater than the deviation of the tubes.

VOL. 5 1961

TABLE 6. COMPARISON OF PERCENTAGE OXYGEN ABSORPTION COMPUTED BY DIFFERENT METHODS

Run No.	Diffusers	Air flow scfm	Percentage Oxygen Absorption				
			Slope method (A)	Moment method	% Variation from (A)	Log method	% Variation from (A)
1	2 saran tubes	13·15	12·22	11·65	−4·66	12·50	+2·29
2	,,	25·70	10·21	9·81	−3·90	10·48	+2·64
3	,,	44·70	8·78	8·93	+1·71	8·85	+0·80
	Average				−2·29		+1·91
4	2 spargers	22·40	5·61	5·55	−1·07	6·15	+8·00
5	,,	35·40	6·72	6·75	+0·04	6·41	−4·61
6	,,	46·20	6·73	7·12	+5·80	6·63	−1·49
	Average				+1·59		+0·63

Fig. 9 shows a typical oxygen absorption curve. The observed dissolved oxygen data for run 6 is plotted and the values of D.O. computed according to the relation $C_L = C_i(1-e^{-K't})$ for the values of C_i and K' obtained by the above three methods are superimposed on this curve. It is apparent there is no deviation from observed values for all points below 50 per cent saturation. This is the range used for computing the rate of oxygen absorption. The log method was adopted as standard because it is much easier to handle in normal computation.

The data in Mr. Dreier's Table 4 provides confirmation of studies at the University of Iowa. The apparent differences in Mr. Dreier's data are due to the effect of tank geometry.

The effect of geometry was studied in the 24 ft wide test tank by operating the tank so that each half of the tank was equivalent to a tank 4 ft by 12 ft by 15 ft deep. This is the same width as the test tank utilized by Mr. Dreier. Fig. 10 is presented to show the effect of reducing the tank width from 24 ft to 12 ft. Since these data are plotted as air flow per tube, one tube in a 12 ft tank must be compared with two tubes in a 24 ft tank to have the same ratio of air flow to water volume. The data of one saran tube in a simulated 12 ft wide tank is compared to two tubes in a 24 ft tank. The absorption efficiencies are similar at low air flows. The improvement of absorption efficiency at higher flows is due to greater bubble carry-over in the smaller tank. All of these data are from tests at the University of

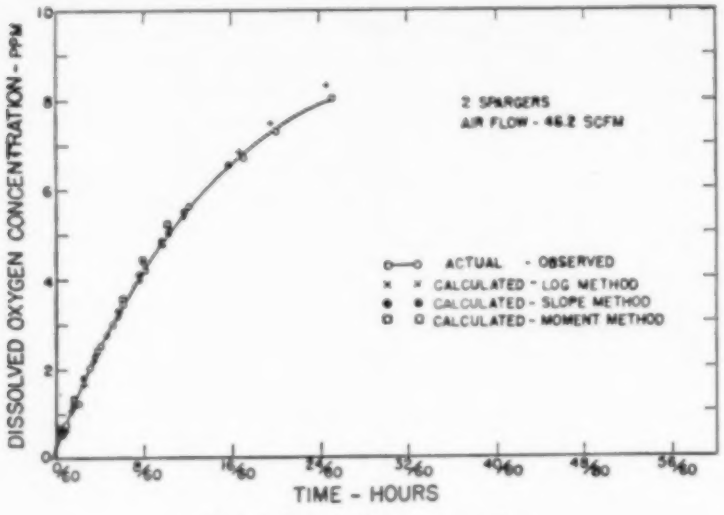


FIG. 9. Dissolved oxygen run 6.

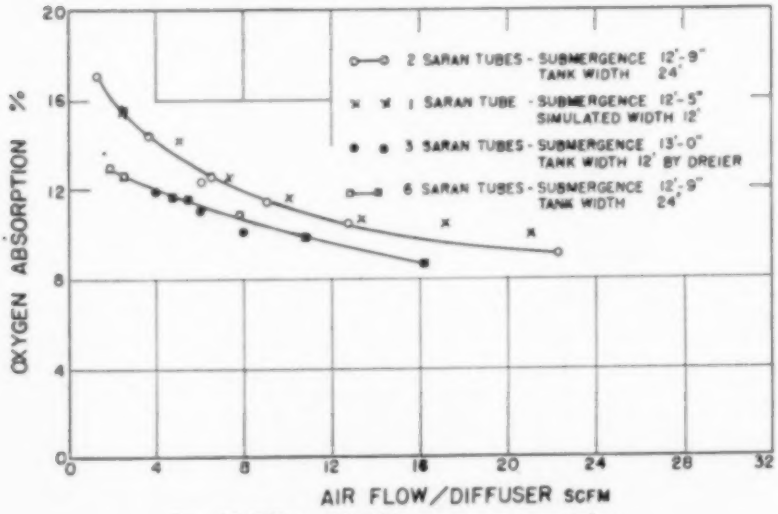

OXYGEN ABSORPTION %
AIR FLOW/DIFFUSER SCFM
2 SARAN TUBES - SUBMERGENCE 12'-9" TANK WIDTH 24'
1 SARAN TUBE - SUBMERGENCE 12'-5" SIMULATED WIDTH 12'
3 SARAN TUBES - SUBMERGENCE 13'-0" TANK WIDTH 12' BY DREIER
6 SARAN TUBES - SUBMERGENCE 12'-9" TANK WIDTH 24'

FIG. 10. Effect of bubble carry-over saran tubes.

Iowa. Fig. 10 also shows Mr. Dreier's data for three tubes in a 12 ft wide tank compared to the Iowa data for six tubes in a 24 ft wide tank. It is apparent that these data are in general agreement which shows that it is possible to check results of various studies with the deoxygenated water method and it clearly demonstrates the importance of tank geometry.

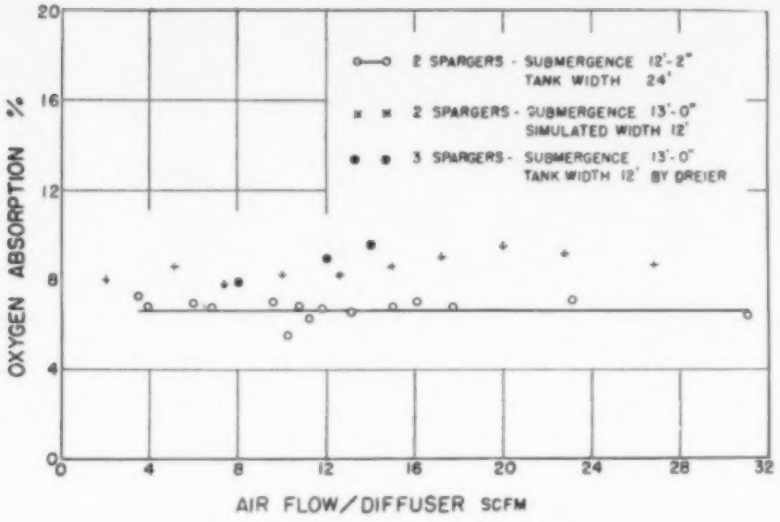


FIG. 11. Effect of bubble carry-over-spargers.

Fig. 11 is similar data for two spargers. Because the oxygen absorption with spargers is directly proportional to submergence the data for the simulated 12 ft tank have been adjusted to 13 ft submergence to be comparable to Mr. Dreier's data. Here again a reasonable close check is obtained between the two studies. The apparent variation was due almost entirely to geometry effect of Mr. Dreier's narrow tank.

VOL. 5 1961

VOL.
5
1961

PLANT TESTS OF SPARGER AND TUBE TYPES OF DIFFUSERS

A. H. CHASICK

Department of Public Works, New York City

THIS paper reports experiences with diffused aeration at the Owls Head, Hunts Point and Rockaway Sewage Treatment Plants. At Owls Head and Rockaway the modified aeration process is employed. Hunts Point employs the step aeration process.

HUNTS POINT

The Hunts Point tests employed saran-wrapped and ceramic diffusers operating at an air rate of 6·5 ft³/min per tube. The clogging characteristics of these units are shown in Fig. 1. The pressure differential is that required above zero flow to

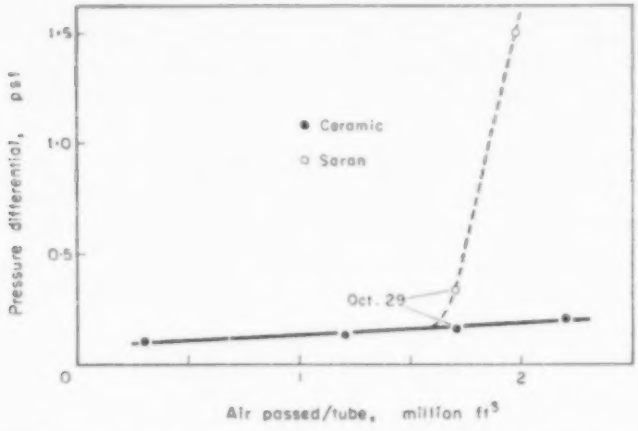


FIG. 1. Hunts Point P. C. P. Pressure differential at 5 ft³/min per tube vs. air passed.

obtain a flow of 5 ft³/min. At the beginning, a differential of 0·1 lb/in² was required. This increased slowly and equally for both the ceramic and saran-wound tubes until the saran-wound tubes began to clog. Then the differential began to increase rapidly until finally the saran-wound tubes could not pass the required air. The ceramic tubes remained virtually unimpaired at this time.

Some of these saran-wound tubes were examined, and it was found that the windings were clogged by dust and soot originating in the adjoining industrial

area. The plant air is filtered through rather inefficient wire mesh velocity-type filters.

A second series of tests employed spargers at an average spacing of 20 in. The spargers used consisted of a 3 in. diameter casting with two internal ducts at right angles to each other, and terminating in four 0·25 in. ports spaced at 90°. They are rated to pass 12 to 18 ft³/min of air each.

Equal quantities of primary effluent and return sludge were applied to two aerators; one equipped with saran-wrapped tubes and the other with spargers. The efficiency of each type of diffuser was compared on the basis of dissolved oxygen only. The sparger appeared to be only two-thirds as efficient as the tubes in transferring oxygen to the mixed liquor under the loading and cold sewage conditions of the tank.

Further tests were conducted using an improved air filter consisting of bags precoated with asbestos. This filter was specified to produce air with not over 0·1 mg of particulate matter per 1000 ft³ of air. The old filters passed air containing from 0·8 to 1·6 mg of dirt per 1000 ft³. The saran tubes clogged after receiving an estimated 2 m ft³ of this air. The new filters reduced the dirt to from 0·06 to 0·20 mg/1000 ft³ and have resulted in no sign of clogging after passing 6·7 m ft³ of air.

OWLS HEAD

At Owls Head, ceramic tubes are still functioning after passing 10 m ft³ of air at a small increase in pressure. A curve indicating the pressure required to pass air at a rate of 6 ft³/min per tube vs. the total amount of air passed is shown in Fig. 2. After almost 4 years of operation, the pressure has increased by about one-third of 1 lb/in².

Saran-wrapped tubes have clogged after passing 2·7 to 4·3 m ft³ each. The aeration liquor tends to foster the growth of slimes because of the fairly high B.O.D. left in solution by the modified aeration process, as compared to step aeration.

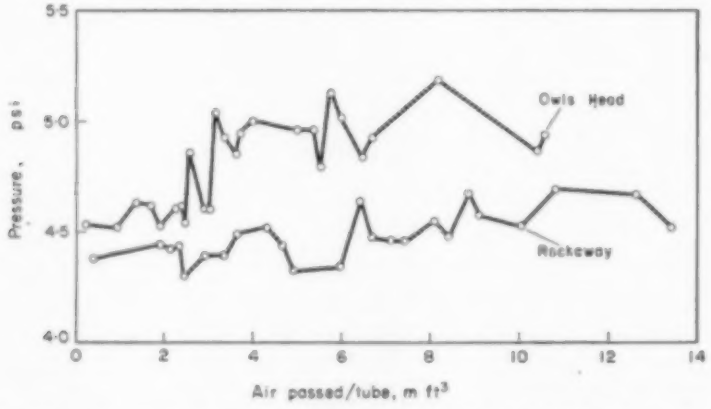


FIG. 2. Rockaway and Owls Head P.C.P. Pressure (at flow of 6 ft³/min) vs. air passed. Ceramic tubes.

ROCKAWAY

At Rockaway only a slight increase in pressure was noted for ceramic tubes after passing some 13 m ft^3 of air. Saran-wrapped tubes have clogged after operation of periods ranging from 70 to 282 days. Experience at the Rockaway Plant indicated that to obtain the same dissolved oxygen in the afternoon, under load, required approximately one-sixth more air for spargers. At Rockaway and the 26th Ward Plants, there was no significant difference, however, in treatment results from the use of tubes or spargers.

It is believed that for small plants the cost of power for aeration assumes less importance and of labor for maintenance greater importance and spargers might be considered as most economical. In large plants however, power costs represent a significant fraction of the total cost and more efficient diffusion devices should be employed.

Acknowledgements—Thanks are extended to the following operating engineers and their staffs: Messrs. A. I. Slocum, Hunts Point: F. H. A. Lino, Owls Head; Leo D. McGeady, 26th Ward; and the late Charles H. Stratton, Rockaway. Members of the sanitary engineering staff who participated in gathering, collating or presenting this data are Wilbur N. Torpey and Martin Lang Chief and Assistant Chief of the Bureau, Norman Nash, Richard T. Dewling and the author.

The author is especially grateful to the late Commissioner Frederick H. Zurmuhlen of the Department of Public Works, whose deep interest in improved techniques made these tests possible, and to William A. O'Leary, Director of the Division of Sewage Disposal, who initiated these tests to obtain information for the better design and operation of New York City's pollution control projects.

VOL.
5
1961

BENCH-SCALE BIO-OXIDATION STUDIES, BATCH VS. CONTINUOUS SYSTEMS

A. W. Busch, Nugent Myrick, K. W. McKinney and J. W. Peterson
Rice Institute

This paper is concerned with the effects of environment on the physiological characteristics of mixed biological cultures as evidenced by clarification and settling properties when grown on a specific soluble substrate, i.e. glucose.

It is a basic concept that for effective utilization in waste treatment a biological culture must be capable of producing a clear effluent and must be amenable to hydraulic separation in order that solids may be returned for continued use. Furthermore, it seems likely that a culture developed under batch conditions may fail when placed in a system of continuous displacement, or that conversely an entirely different culture may be developed on the same waste in different hydraulic and food supply environments. Certainly batch systems fed once a day should not be expected to attain the steady state growth of optimum oxidation efficiency.

VOL.
5
1961

PROCEDURE

The system employed for continuous treatment and for partial treatment has been fully described in a previous publication (1). The substrate utilized was glucose supplemented appropriately with nitrogen and phosphorus. Tap water was used in the preparation of the continuous unit feed solution and to replace the wasted volume from the batch systems in order to insure the presence of trace nutrient elements.

Two batch systems were set; one for 15 per cent and the other for 50 per cent daily wastage of mixed liquor. The continuous unit timers were set for wastage rates ranging from 15 to 75 per cent and, including effluent solids carry-over, total wastage ranged from 65 to 115 per cent daily. Both units had same total feed.

EXPERIMENTAL RESULTS

The character of biological solids as measured by clarifying ability and B.O.D. removal differed markedly in the two types of systems. While the median effluent soluble B.O.D. of the continuous systems was consistently below 10 mg/l. the filtrate B.O.D. of the wasted mixed liquor from the batch systems was much higher. Fig. 1 shows that in the 15 per cent wastage system the filtrate B.O.D. increased throughout the 77 day test period and reached a value more than twice the initial feed concentration. The 50 per cent wastage system appeared to reach an equilibrium of sorts and the B.O.D. varied between 20 and 400 mg/l.

Fig. 2 shows consistent solids–growth–age relationships for the two systems and indicates similar population growth characteristics. Calculations of solids

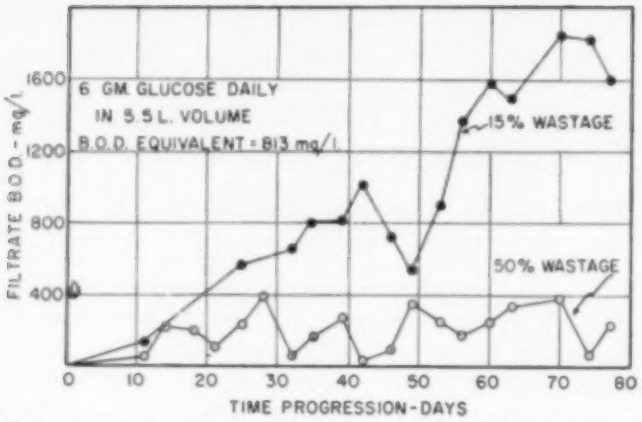


FIG. 1. B.O.D. characteristic of filtered supernatant from batch fed bio-oxidation units.

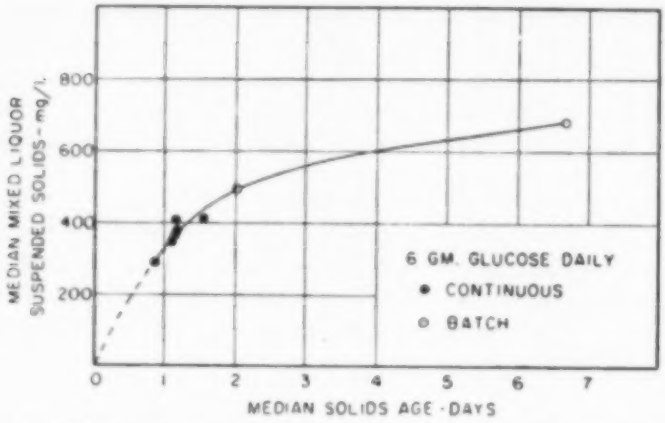


FIG. 2. Solids-growth-age relations in completely mixed bio-oxidation systems.

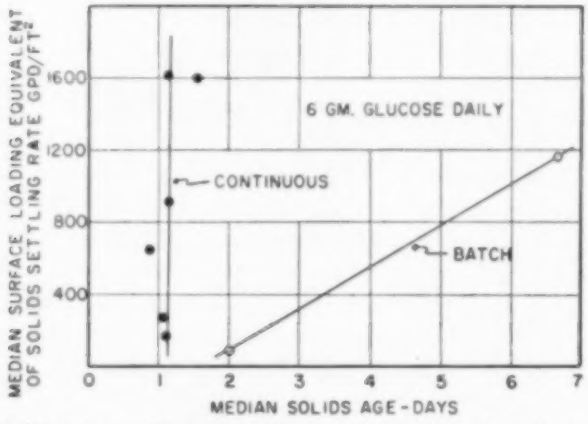

MEDIAN SURFACE LOADING EQUIVALENT OF SOLIDS SETTLING RATE GPD/FT²
6 GM. GLUCOSE DAILY
CONTINUOUS
BATCH
MEDIAN SOLIDS AGE - DAYS

FIG. 3. Effect of solids age on solids settling property in completely mixed bio-oxidation systems.

produced per gram of B.O.D. removed also show consistent results in the two systems.

The solids settling characteristics for the two systems varied over about the same range even though no hydraulic requirement existed in the batch units. The significant characteristic of this parameter is the variable of which the settling prop-

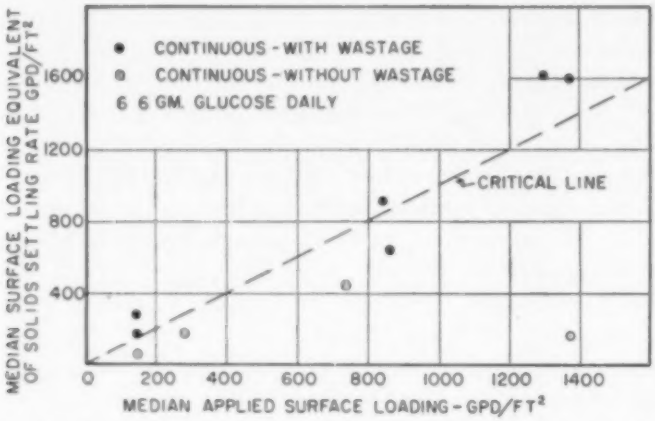


FIG. 4. Effect of surface loading on solids settling property in completely mixed bio-oxidation systems.

VOL. 5 1961

erty is a function. Fig. 3 shows settleability of solids in the batch system to be a function of solids age as was expected from previous batch system experiments. However, the solids of the continuous system are independent of culture age and are a function of the applied surface loading as shown in Fig. 4. In order to show that simple washout is not the sole cause of this relationship, data for a continuous system operated without wastage are included in Fig. 4. Significantly the only point in the continuous-with-wastage system which fell below the critical line represented a timer induced wastage of 77 per cent and a total wastage (including washout) of 115 per cent daily.

DISCUSSION OF RESULTS

It is emphasized that although the results of this study should be relative on the broad scale, specific relationships delineated herein must be restricted to glucose-grown solids. Glucose characteristically produces a fluffy, lightweight biological growth. By increasing the solids age the density can be markedly increased but this is possible only on a batch basis wherein no hydraulic loading requirement is imposed.

Glucose was selected as the substrate for this investigation for several reasons. First, a considerable amount of work involving bench-scale oxidation studies with glucose as substrate has been published by other workers. Second, the solids growth characteristics are being studied with respect to the B.O.D. theory recently proposed

(3) and third, soluble substrates are believed to present the most difficult problem in industrial waste treatment.

The results reported tend to substantiate conclusions reached by other workers regarding the effect of high growth rates (low solids age) on process efficiency and sensitivity. Theoretically the bacterial population required to remove a given amount of B.O.D. in a fixed time may vary within wide limits. The one extreme is a young, actively growing population of relatively few numbers whose metabolism is keyed to high synthesis since the base food demand is low. At the other extreme is a large population which requires the predominance of the available food for maintenance of its large numbers and hence synthesizes only enough organisms to balance those lost by normal decay rates. Loss of dead cells in the effluent from a high solids concentration operation often produces a lower quality of treatment. Extension of high solids systems to operation without intentional solids wastage has been proposed as the so-called total oxidation process. Recent articles have demonstrated the fallacy of such operation (2) (4) (5).

The limits of practicable solids age are probably dependent on the substrate. However, studies made to delineate these limits for a particular waste must be conducted in an environment similar to the anticipated full scale operation. Garrett (6, 7) has shown how growth rates, and hence solids age, may be theoretically and practically controlled. The present study has shown that batch procedures employing once a day feeding and wasting will not simulate continuous completely mixed operation even though solids production per unit of B.O.D. removed are identical for the two systems. The steady-state equilibrium produced by continuous food supply, continuous solids wastage and continuous hydraulic loading cannot be duplicated by batch operation.

SUMMARY

Results of a comparative study of batch and continuous completely mixed systems operated on glucose as substrate have been reported. Batch type studies are useful for determining basic susceptibility to biological oxidation, maximum growth rates attainable and solids production per unit of substrate oxidized. However, the design of a completely mixed continuous flow system on the basis of mixed liquor solids concentration, B.O.D. removal and hydraulic separation characteristics of batch grown solids is hazardous. Settling characteristics of batch grown solids are a function of solids age while in continuous systems the applied surface loading determined the settling characteristics of the solids.

Acknowledgement—A portion of this work was supported by N. I. H. Grant R. G. 5534.

REFERENCES

1. BUSCH, A. W., Theory and design of bench-scale units for biological oxidation studies. *Wat. Sewage Wks.* **106**, 6, 254 (1959).
2. BUSCH, A. W. and MYRICK, N., Food-population equilibria in bench-scale bio-oxidation units. *J.W.P.C.F.* **32**, 9, 949 (Sept. 1960).

3. Busch, A. W., B. O. D. progression in soluble substrates. *Sewage Industr. Wastes* **30**, 11, 1336 (1958).
4. Symons, J. M. and McKinney, R. E., The biochemistry of nitrogen in the synthesis of activated sludge. *Sewage Industr. Wastes* **30**, 7, 874 (1958).
5. Kountz R. R. and Forney C., Jr., Metabolic energy balances in a total oxidation activated sludge system. *Sewage Industr. Wastes* **31**, 7, 819 (1959).
6. Garrett M. T., Jr. and Sawyer C. N., Kinetics of removal of soluble B.O.D. by activated sludge. *Proc. 7th Ind. Waste Conf. Purdue University, 1952.*
7. Garrett M. T., Jr., Hydraulic control of activated sludge growth rate. *Sewage Industr. Wastes* **30**, 3, 253 (1958).

DISCUSSION

Ross E. McKinney: In their paper the authors have tried to delineate some of the problems between batch fed and continuous fed activated sludge systems. The data that they have presented clearly showed that there was no comparison between batch fed systems and continuous fed systems on a laboratory basis. Yet, if the reader is not careful, he may draw the conclusion that batch fed data and continuous flow data can never be the same. This is not the case, as it is possible to obtain duplicate data on batch systems as on continuous flow systems.

In their systems the authors were not fair to the continuous fed system during early sludge build-up. A hydraulic overflow rate was imposed on the system of 1440 gal/ft² per day from the beginning of operation when solids in the aeration section were low. This hydraulic overflow rate caused the light microbial floc to be carried out in the effluent and prevented a solids build-up. On the other hand, the batch fed system had a hydraulic overflow rate of 0 gal/ft² per day such that the microbial floc had the maximum opportunity for settling and hence retention in the system.

In working with continuous flow systems in the laboratory and in full scale installations it is necessary to build up a heavy activated sludge so that the continuous operation does not cause the floc to be blown out of the sedimentation system. If the authors had built up a floc to a reasonable initial concentration, 1000–2000 mg/l., they would have found that both systems would have operated identically with regard to effluent quality and solids build-up. The problem of sludge build-up is of considerable importance in field installations since it is almost impossible to build up a heavy sludge mass from a small M.L.S.S. concentration with high hydraulic flow rates.

Laboratory continuous flow systems can be used for determinations of treatability of wastes, as well as determination of design criteria provided that the data is properly translated for field conditions. Under identical conditions in the laboratory and in the field, the same results will be produced in both systems. This was amply demonstrated in the design and operation of the waste treatment system for HoHoKus Bleachery,* a 125,000 gal/day plant built from data obtained on a 1·5 l. continuous flow pilot unit. The data from the two systems checked within 2 per cent of each other.

* McKinney, R. E., Symons, J. M., Shifrin, W. G. and Vezina, M., Design and operation of a complete mixing activated sludge system, *S&IW* **30**, 287–95 (1958).

VOL. 5 1961

PREDICTION OF THE PERFORMANCE OF COMPLETELY MIXED CONTINUOUS BIOLOGICAL SYSTEMS FROM BATCH DATA

ROY F. WESTON and VERNON T. STACK, JR.

Roy F. Weston, Inc.

BATCH laboratory scaled studies are subject to all of the inherent difficulties of biological oxidation systems with the added complexities of large surface-to-volume ratios, small quantities of mass to deal with, and magnified effect of toxicity or other undesirable factors because of "slug" application of food material.

Regardless of inherent difficulties, batch studies of biological oxidation remains an attractive approach to the problem for several reasons. In the absence of toxicity problems, batch studies offer a relatively speedy technique for the development of fundamental information. This information can be used as a guide in defining the scope of pilot-scaled studies, or, if the investment in the final facility is so small that a pilot study cannot be justified, carefully determined laboratory-scaled data may be used for design purposes. The concepts of initial removal, biological synthesis and equilibrium have been previously developed for batch oxidation systems (1).

VOL.
5
1961

PREDICTION OF CONTINUOUS PERFORMANCE FROM FUNDAMENTAL BATCH DATA

Translation of initial removal

Initial removal of B.O.D. is a rapid reaction, and absence of the time element simplifies translation of batch data to the continuous system. The fundamental assumption is that the variable product fb determined from batch data applies to the continuous system. The initial concentration of B.O.D., L_0, is calculated from B.O.D. concentration in the feed and the B.O.D. concentration in the return sludge. The initial concentration of volatile solids, S_0, is calculated as that which is established by mixing the feed and returned sludge. Initial removal is then calculated as

$$L_{ri} = fbL_0S_0$$

This calculation also can be applied to activated sludge systems which have internal sludge return. The value of S_0 is the volatile solids concentration in the aeration mixture, and L_0 is the concentration of B.O.D. in the feed since there is essentially no hydraulic return.

Efficiency of B.O.D. removal

A typical B.O.D. remaining versus time curve for batch treatment is illustrated in Fig. 1. The data in the observed form does not represent a completion of reaction curve from which performance in a continuous system can be predicted directly.

A large amount of B.O.D. remaining at equilibrium in the batch system might appear to be a limiting factor in activated sludge performance. Equilibrium in the removal of B.O.D. is assumed to represent the condition of pure auto-oxidation. The rate of B.O.D. removal from the substrate for biological synthesis is equal

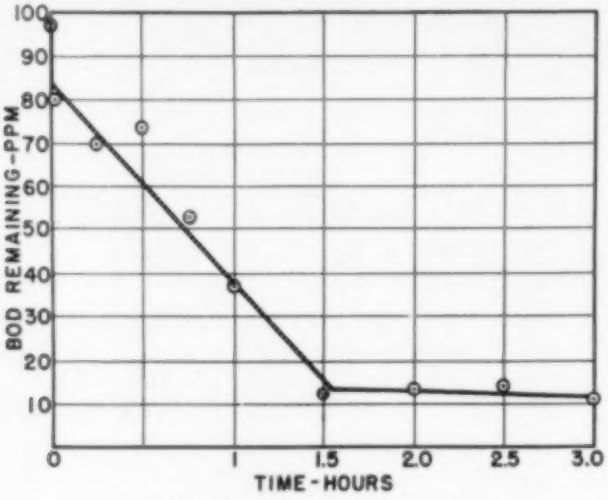


FIG. 1. B.O.D. removal in a batch system.

to the rate of B.O.D. released by lysis and auto-oxidation. Nevertheless, completely mixed continuous systems fed the same waste will accomplish treatment efficiencies much higher than those observed in the batch system at equilibrium.

Many observations of batch activated sludge treatment of wastes in our laboratories have revealed that the amount of B.O.D. remaining at equilibrium is a function of the initial B.O.D. in the aeration mixture only. It is a constant percentage of the initial B.O.D. of the mixed liquor. Thus, if the feed application is controlled to establish a very low initial B.O.D., the B.O.D. remaining at equilibrium theoretically can be reduced to a negligible quantity. Therefore, plant efficiency measured from feed to effluent can approach 100 per cent.

This observation suggests that performance in a batch activated sludge system from start to equilibrium reflects the resultant of B.O.D. transfer rates. If the assumption is made that the same transfer rates occur in a completely mixed continuous system and that the same resultant of rates occurs, B.O.D. removal in the continuous system can be controlled by some mean transfer constant predictable from the resultant rates observed in the batch system.

The driving force involved is not easily defined because B.O.D. transfer in batch activated sludge systems does not appear to correlate as a first-order reaction.

Although a first order correlation is not apparent, correlation of batch and continuous data was attempted by a first order approach. It was assumed that the rate of B.O.D. removal observed in the batch system is the resultant of a driving

force related to the amount of food present. If this is accepted, the controlling B.O.D. transfer coefficient could be equal to the rate of transfer divided by some function of the B.O.D. present.

Correlation of batch and continuous data gave a reasonable fit when the apparent rate of B.O.D. transfer was divided by the average B.O.D. present at equilibrium,

$$r = \frac{L_i - L_e}{t_e L_e},$$

where

L_i = B.O.D. remaining after initial removal ($L_0 - L_{ri}$) (ppm),

L_e = B.O.D. remaining at equilibrium (ppm),

t_e = time required to reach equilibrium,

r = the apparent coefficient of mass transfer.

The transfer constant calculated in this manner assumes that the continuous system is operating at the equilibrium point in the batch system. The predicted results using this constant are optimum and may be difficult to reach. Several factors can prevent equilibrium from being established such as, degree of mixing, fluctuations in feed strength or pH, variations in the types of materials in the feed, or minor biostatic effects.

VOL. 5 1961

Where pilot-scaled continuous data are developed a transfer constant can be calculated from batch data which will account for conditions that prevent the establishment of equilibrium conditions. This possibility will be illustrated in the discussion of actual results.

Treating the apparent coefficient of B.O.D. transfer as representing a first-order transfer, the degree of completion of reaction with time is given by equation

$$D = 1 - (1 - r)^t$$

where

D = fractional completion of reaction in time t.

In the application of this equation, care must be taken to avoid a mathematical pitfall. The equation provides an approximation of a first-order curve and the best approximation is obtained with small values of the integral, r. A practical maximum value of r is 0·2. It is generally convenient to calculate r on a per hour basis. Where r per hour is a large number, it may be reduced to some fraction of an hour to provide a usuable value of r.

The fractional completion of reaction equation and mixing theory can be used to predict B.O.D. removal in a continuous system. This is accomplished by determining the treatment age of each fraction of effluent discharged from the process. By summation of treatment for the individual fractions, the total amount of treatment is predicted.

For example, assume that for the desired treatment conditions the value of r equals

0·10 per hour and the flow to the aeration basin is such that the mean flow-through period will be 10 hr. The fractional completion of reaction curve is shown in Fig. 2.

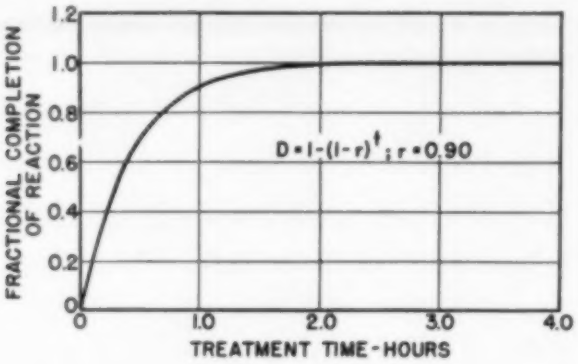


FIG. 2. Reaction rate curve calculated from estimated B.O.D. transfer coefficient.

A mixing theory curve for a single basin is shown in Fig. 3.

By selecting a treatment time and calculating t/t_b the portion of the effluent which will be held in the aeration basin for any given treatment period or longer

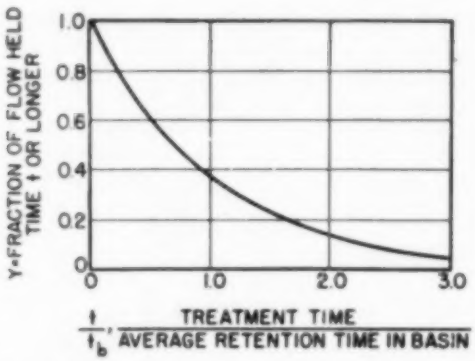


FIG. 3. Holding times in a completely mixed tank.

is obtained from Fig. 3. The degree of completion of reaction for any treatment period is obtained from Fig. 2. Tabulated results are shown in Table 1.

The quantity $1-Y$, in Table 1 represents the portion of the flow which is held in the aeration tank for a period of time equal to or less than t when the mean flow-through time is 10 hr.

In a plot of D versus 1–Y, the ratio of the area under the curve to the total area above and below the curve is an approximation of fractional completion of reaction in the continuous system. For the particular example, plotted in Fig. 4, the area under the curve is 0·513 of the total area. Therefore, predicted B.O.D. removal

TABLE 1. EXAMPLE OF MIXING THEORY CALCULATION
($t_b = 10$ hr, $r = 0{\cdot}10$).

t	t/t_b	Y	D	$1-Y$
20	2·0	0·135	0·88	0·865
15	1·5	0·225	0·80	0·775
10	1·0	0·37	0·65	0·630
7·5	0·75	0·47	0·55	0·530
6·25	0·625	0·535	0·48	0·465
5·0	0·5	0·61	0·41	0·39
3·75	0·375	0·69	0·33	0·310
2·5	0·25	0·78	0·23	0·220
1·25	0·125	0·88	0·13	0·120
0	0	1·00	0	0

by the continuous system due to synthesis of new cell material is 51·3 per cent of the initial B.O.D.

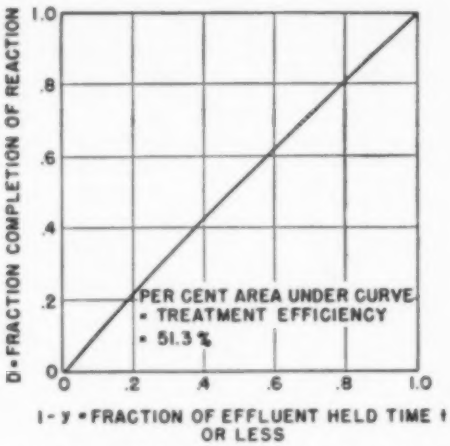


FIG. 4. Graphical integration of mixing theory data to determine treatment efficiency.

If initial removal of B.O.D. is significant the total prediction of performances must include initial removal. The per cent efficiency becomes

$$\text{per cent B.O.D. removal as process efficiency} = \frac{L_{ri} + [L_i \times (\text{predicted fractional completion of reaction})] \times 100}{L_0}.$$

Predictions of continuous performance from batch data by mixing theory provides a good first approximation within the accuracy of analytical measurements in biological systems. Correlations within a group of activated sludge pilot data are shown in Fig. 5.

VOL. 5 1961

Some of the spread in these results is due to a wide range of operating conditions over which *fb* valves were not accurately known. However, the average line of the prediction is close to the average line of the data.

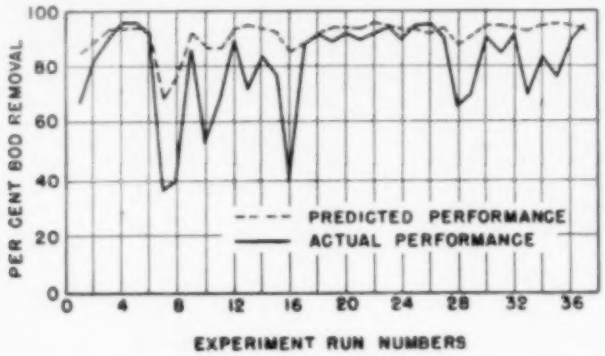


FIG. 5. Prediction of pilot plant performance.

The dashed curve represents the predicted performance which assumes primarily that equilibrium conditions exist. The major deviations from predicted performance can be related to factors which disrupted equilibrium conditions.

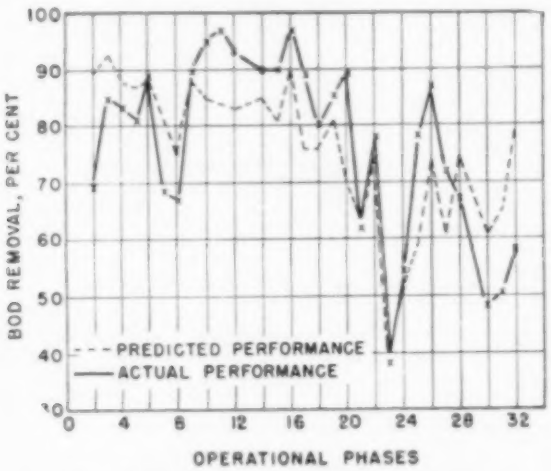


FIG. 6. Prediction of pilot plant performance.

Correlations between plant scale results and predictions from batch data are shown in Fig. 7 for a Stabilization Basin operation. The continuous plant results are widely scattered because the batch treatment data included a two-fold variation in treatment rate. The demonstration of variability in treatment rate alone can justify the use of these techniques.

The stabilization basin and batch data correlated best using a rate constant calculated as

$$r = \frac{L_i - L_e}{t_e} \Big/ \frac{L_i + L_e}{2}.$$

This correlation was the result of less than optimum mixing and the presence of minor biostatic conditions which prevented the continuous system from operating at the batch equilibrium.

The results of other activated sludge pilot plant data are shown in Fig. 6. The predicted curve is based on fb relationships and rate constants determined from the average of a few batch-scaled studies. Deviations of the actual results from the predicted can be explained by the various operational phases of the pilot-scaled study.

Through phase nine the plant was operated as conventional activated sludge without sludge stabilization. For this part of the study the predicted curve is higher because it predicts more initial removal than actually occurred.

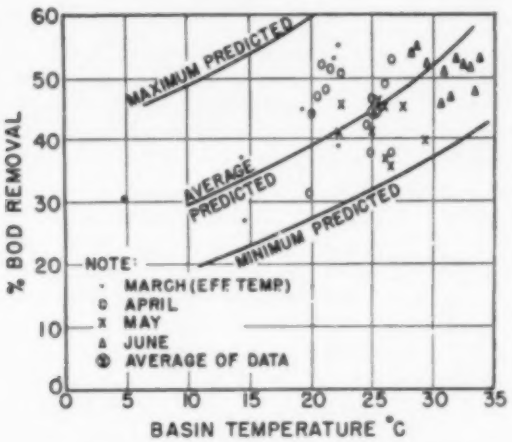


FIG. 7. Stabilization basin treatment efficiency.

During phases nine through twenty, the plant operated under optimum conditions as a contact-stabilization process. Actual performance was greater than predicted, and again, initial removal was the important factor.

At the end of the study, the plant was operated with inadequate oxygen supply which prevented optimum performance.

SUMMARY

Batch data for the stabilization basin and activated sludge processes can be used to define the scope of pilot-scaled studies on completely mixed systems and may in numerous instances be used for design purposes. However, if the waste involved is biostatic or toxic, evaluation of activated sludge performance through the batch

VOL. 5 1961

approach should not be considered. For suitable wastes, batch studies can be used to determine fundamental information.

Initial B.O.D. removal is essentially an instantaneous reaction. A direct translation of batch initial removal data to a continuous system appears to be practical.

B.O.D. removed by synthesis reactions cannot be translated directly from batch to continuous systems. A reasonable accurate prediction of performance by a completely mixed system can be made by calculating an apparent B.O.D. transfer coefficient from batch data. A fractional completion of reaction curve calculated from the transfer coefficient can be combined with mixing theory to predict performance in a continuous system.

REFERENCE

1. Weston R. F., The art and science of activated sludge. *Proc. 8th Southern Municipal and Ind. Waste Conf.*, April 2–3, 1959, p. 119.

DISCUSSION

Ross E. McKinney: Sanitary engineers have long sought methods to convert laboratory data to full scale installations. The difficulties manifested in such translations have been found in the large number of treatment plants which have failed to operate as predicted from the laboratory study. The authors have tried to concern themselves with one method for translating batch fed activated sludge laboratory data into full scale continuous flow activated sludge plants.

Having designed a number of complete mixing activated sludge systems, I cannot help but ask the question of why is it necessary to translate batch data to continuous flow complete mixing systems? In the past 5 years of working with completely mixed, continuous fed systems, I have found that the continuous fed, completely mixed laboratory activated sludge systems yield the best data for translation to full scale continuous fed, completely mixed activated sludge plants. At no time has there ever been any problem with regard to loss of M.L. S.S or to loss of microbial activity in the aeration section.

There are certain problems which must be considered in the translation of laboratory data to full scale data. One of the prime problems which has caused most of the trouble to sanitary engineers is the failure to recognize that it is possible to transfer oxygen at a faster rate in laboratory activated sludge units than in the field. The rate of waste feeding to the laboratory unit must be adjusted to the oxygen transfer capabilities of field equipment. Once this has been done, the biological pattern of the two systems will essentially be identical provided the microbial solids can be separated from the liquid. The loss of suspended solids in conventional sedimentation tanks is sufficiently well established that the pilot unit can be adjusted by wasting sludge to equal the solids lost in the field unit. In this way the sedimentation section of the laboratory unit does not need to be operated in the same fashion or at the same overflow rate as in the field. Basically, a laboratory activated sludge unit is operated merely to demonstrate biological treatability since the continuous fed, complete mixing activated sludge system yields a constant design criteria for any given set of operating conditions regardless of the nature of the waste. A given organic load for domestic sewage will give the same results as a phenol waste or a sugar waste under the same environmental conditions provided corrections are made for any inert solids in the wastes. The oxygen demand will be the same, the solids production will be the same and the effluent quality will be the same as long as good microbial separation is possible.

Thus, it is that when data is desired for continuous fed, complete mixing activated sludge systems, it can best be obtained from laboratory units operated continuously on the completely mixed principle. The size of the pilot plant is insignificant in the biological design of such systems since identical results can be obtained in 1·5 l., 1·5 gal, 15,000 gal, 150,000 gal or any volume provided the environmental conditions in all systems are equal.

AUTHOR: There is general agreement with Dr. McKinney's comments when applied loosely to the performance of biological oxidation systems. The continuously fed laboratory system does provide excellent data which translate directly as a design basis for full-scaled facilities. Where toxicity is a problem, continuously fed systems provide the only practical approach to the development of design data. The laboratories of Roy F. Weston, Inc. routinely use continuously fed systems, and the results have been excellent.

Dr. McKinney has left a great deal unsaid which is pertinent and does much to qualify his general conclusions. First, it must be realized that Dr. McKinney is discussing activated sludge systems which are lightly loaded per unit of biological solids. His general approach is to say that oxygen transfer rates greater than 40 ppm per hr are impractical, and this must be a limiting design consideration. In using this basis, he is assuming that oxygen transfer will be accomplished by air blown into the aeration mixture through diffusers. By making this limitation, he accomplishes the following:

(a) The process development effort is minimized because of the limited performance conditions.

(b) The treatment period is long. Hence, per cent B.O.D. removals will be similar even when the actual reaction rates for different wastes may be considerably different. When time is infinite, all reactions are completed independent of the reaction rate.

Dr. McKinney's approach may or may not provide the best answer to the waste treatment problem. The development of continuous system data which permit full evaluation of process economics requires considerable time for empirical evaluation of possibilities much broader than those which Dr. McKinney normally selects.

The treatability and the fundamental relationship, such as oxygen requirements, rates of mass transfer, balance of auto-oxidation and sludge production differ widely. Dr.McKinney mentions sewage, phenol and sugar as if they represent a broad selection of biological oxidation problems. Actually, these are easily treated materials. Others, such as chlorinated organics, propylene glycols, branched alkoxy derivatives of alcohols and acids, and certain paper mill wastes, behave differently, and the resulting fundamental design data vary widely.

Batch biological oxidation systems have a definite place in activated sludge process development studies because:

(a) Time is saved, which means that the laboratory is more productive and saves money for the client.

(b) The fundamental data are developed directly instead of empirically, and the results translate accurately to continuous performance.

(c) Certain important information, such as initial removal is obtained in batch studies and is not obtained readily in continuous studies.

VOL. 5 1961

THE BEHAVIOR OF FILTER BIOTA UNDER CONTROLLED CONDITIONS

WILLIAM T. INGRAM and GAIL P. EDWARDS
New York University

CONTROLLED application of settled sewage and air to an experimental filter 18 ft deep have provided conditions favorable to the growth of trickling filter organisms with organic loadings exceeding 20,000 lb A.F.D. and hydraulic loadings up to 439 mgad. Studies of the microbiota of the filter, the apparent habitat of different groups of organisms, their occurance under various loading conditions, and possible relationships between degree of treatment achieved and the presence of difference organisms at various filter depths are discussed. Some information on the presence of coliform organisms and of bacteria shown by plate count at 20°C to be present at different levels of the filter are presented.

The process of controlled filtration has been described in earlier papers (1, 2). However, it is desirable to review briefly the essential features so that the differences between the application of waste water and air under controlled conditions and the processes known as standard and high rate filtration are understood.

Sewage is taken from the street sewers and after passing it through a settling tank having approximately one hour detention it flows to a tank which is used as a common source of settled sewage for the experimental unit. Two filter columns are shown. These are independent filters. One has been operated at a hydraulic rate of 2·55 gal ft² min for more than 2 years. It has served as a base line for comparisons with the second filter under a variety of operating conditions. The second filter has been loaded at hydraulic loadings up to 7 gal ft² min with settled sewage from the same source taken simultaneously with sewage applied to no. 1 filter. The sewage has been applied entirely at the top and also in steps with 5·09 gal ft² min applied to the top and 1·91 gal ft² min applied to the second section.

Each experimental filter is 18 ft deep and is constructed as a series of independently supported 3 ft deep sections. The sections are connected with a seal which provides for a continuous downward flow of treated water from section to section. The casing is air-tight from top to bottom and air admitted to the under side of each section flows upward and out the top of the filter column. Metered quantities of air are applied to each section by a blower fan. Air pressure within the filter is less than 2 in. H_2O head, as there is ample channel space in the void space to allow reasonably free flow of the quantity of air supplied.

The filter has never clogged. The flow rate on filter no. 1 is 20·4 ft/hr and on no. 2, 56·15 ft/hr. These rates compare to an areal loading of 160 and 437 mgad, respectively.

The process described is a single pass treatment of waste water. There is no recirculation of effluent.

VOL. 5 1961

When step feeding is practiced, hydraulic loading increases as each increment of wastes is added. This in turn increases the sectional loading of B.O.D.

While untreated waste is being added to the first or top sections, dissolved oxygen levels are not increased, as all of the oxygen appears to be utilized immediately by an active aerobic biota. In lower sections of the column, the dissolved oxygen increases and effluent dissolved oxygens are in the range of 2 to 3·5 mg/l.

During January and February, 1960, no. 2 filter has had an average organic loading of 18, 200 lbs A.F.D. (418·6 lbs/1000 ft^3) with an average efficiency of 62 per cent. The effluent B.O.D. has averaged 32 mg/l. Some special studies during this period have indicated that with a settled sewage loading of 20, 150 lbs A.F.D. (463·5 lb/1000 ft^3) the B.O.D. removal was 68·9 per cent.

Some information has been obtained on the relationship of B.O.D. of mixed, settled and centrifuged settled sewage. It was indicated that the B.O.D. of the centrifuged waste was approximately 67 per cent of the total B.O.D. Using centrifugal samples, the B.O.D. removal through the filter was about 63 per cent under the loading conditions described. Analyses of thoroughly shaken settled sewage and filter effluent samples indicated a removal in the filter of about 53 per cent. It appears that filter solids in themselves account for about 20 per cent of the B.O.D. since the overall efficiency of B.O.D. removal from settled sewage in treated centrifuged samples was 75 per cent. Solids determinations established that few suspended solids remained after centrifuging.

At these loadings NO_2 nitrogen was minimal averaging 0·056 mg/l. Total organic nitrogen reduction through the filter amounted to 75 per cent with an average of 5·5 mg/l. present in the settled waste in January and February, 1960.

Coliform testing is continuing. During the first 3 months of the year the log average M.P.N. showed an increase in number with the record as follows:

	M.P.N.
Settled effluent	12,500/ml
No. 1	73,300
No. 2	47,800
No. 3	55,600
No. 4	49,400
No. 5	41,700
Effluent	55,200

These data represent the results of seventeen samplings. All samples were settled 30 min before taking portions for analysis. It may be inferred that there is some indication of a favorable habitat for coliform within the filter. This in turn offers some interesting thoughts concerning the possibility of coliform growth in the presence of partially stabilized organic materials and a reasonably clean water. These data also open avenues of thinking concerning what is being measured when coliform density of treated sewage is used as an index of pathogenic potential. In this instance a four-fold increase in coliform surely cannot be interpreted as a like increase in the occurrence of pathogenic organisms.

Some studies were made of bacterial populations growing on agar at 20°C. These data were so erratic that there is little that can be said except that the counts ranged from less than 100,000/ml to nearly 700,000 in the settled sewage and from 60,000 to 900,000 in the effluent. There is indication of prolific and continuing growth of bacteria throughout the filter even though protozooans and higher forms may utilize bacteria as a source of food.

Filter organisms tend to have a location in the filter that is more favorable for their growth. Flagellates are seldom found in the first section whereas zooglea are usually found even though their growth is not prolific. The first section appears to be most suited for free swimming ciliates and worms. This and the second section show debris. Filamentous bacteria also appear in this section.

Occasionally stalked ciliates are found along with rotifers and fungi on the first section. These ciliates become prolific in the lower sections. Zooglea tend to increase and are predominant in the third and fourth sections then become less prevalent in the sixth. Free swimming ciliates are found occasionally throughout the filter, but seldom do they appear in quantity. Worms are scattered throughout.

Rotifers seem to grow more abundantly in the last three sections. Fungi may occur in some quantity in all sections. Flagellates seem unimportant in the filter and are found only occasionally in the lower sections.

In appears that stalked ciliates and rotifers need a reasonably well purified oxygen-containing water to grow well. Their presence at times in upper sections can usually be related to a weak sewage or to an increased efficiency of B.O.D. removal in an upper filter section.

Research into biota habitat is continuing and we hope to find reasons for the indicated selectivity. Experimental work is now being set up in which we can study the growth of organisms on media other than stone—materials of definite patterns for which the specific media surface can be estimated with reasonable accuracy.

Acknowledgement—This research has been supported in part by a Research Grant from the National Institutes of Health. Appreciation is expressed for the work of John Bevan and others of the staff who keep the experimental unit working under difficult operating conditions.

VOL. 5 1961

REFERENCES

1. INGRAM W. T., An approach to the design of trickling filters. *Trans. Amer. Soc. Civil Engn.* **122**, 1202 (paper no. 2905) (1957).
2. INGRAM W. T., An investigation of the mechanisms of controlled filtration. *Sewage Industr. Wastes* **31**, 10 (1959).

TRICKLING FILTER FORMULATIONS

V. C. BEHN

University of Delaware

Abstract—An attempt is made to correlate the various theories of trickling filter operation which are based on a mathematical model of the filter.

Particular attention is given to the work of Phelps, Velz, Schulze, Howland and Stack. The mathematical aspects of each analysis is given attention in order to bring underlying assumptions into focus. Special emphasis is placed on reaction kinetics and the role of recirculation.

The paper concludes with a discussion of the type of experimental work which might serve to help clarify some of the variance in the theoretical analyses.

INTRODUCTION

A NUMBER of attempts have been made to develop a mathematical model for the trickling filter process in waste treatment. Any individual who desires to consider these various models in development work is faced with the problem of co-ordinating material from the literature. This paper is an attempt to aid this process by discussing the various models and their derivation in some detail.

VOL.
5
1961

In particular, an attempt is made to demonstrate the relationship between the equilibrium load contained in a saturated filter as proposed by Phelps and Velz, and the applied load which will saturate a unit depth as suggested by Stack. Also, the availability factor utilized in the National Research Council formula is reviewed from the standpoint of its physical interpretation and its role in the Stack equation for a recirculating filter. This work, which uses depth as a parameter, is compared with that by Howland which utilizes time of contact. The work of Schulze utilizing Howland's relationship between time, depth and flow rate is also considered.

As the title of this paper implies, the scope is directed towards the theoretical aspects of trickling filters. This is not intended to convey a lack of appreciation for the empirical aspects so important to the design engineer. In fact, the theoretical equations are based upon empirical assumptions. Attention to the details of the mathematical analysis is necessary in order to bring the underlying assumptions into clearer focus.

The theoretical aspects are straightforward, usually involving reaction kinetics and series, the latter being introduced when recirculation is considered. Accordingly, discussion in this paper will be in terms of these two topics. The paper concludes with a summary statement which emphasizes those aspects of the theoretical analysis which would benefit by additional experimental verification.

REACTION KINETICS

The rate of purification of organic matter in a biological filter is given by Fair and Geyer (1) as

$$dL/dt = -K_t(L/L_a)^n L, \tag{1}$$*

or alternatively

$$dL/dD = -K_D(L/L_a)^n L \tag{2}$$

In these equations L represents the organic load in terms of B.O.D. equivalent. The exponent n expresses the deviation from a first-order process, the entire group $(L/L_a)^n$ introducing a factor which relates the purification rate to the degree of purification which has taken place. When $n = 0$, these equations reduce to the familiar form of the first order reaction

$$dL/dt = -K_t L, \tag{3}$$

or alternatively

$$dL/dD = -K_D L. \tag{4}$$

Integration of equations (1) and (2) is straightforward, involving separation of variables. The result is

$$L_t/L_a = [(1/(1+nK_t t)]^{1/n}, \tag{5}$$

or alternatively

$$L_D/L_a = [(1/(1+nK_D D)]^{1/n}. \tag{6}$$

Integration of equations (3) and (4) yields

$$L_t/L_a = \exp(-K_t t), \tag{7}$$

or alternatively

$$L_D/L_a = \exp(-K_D D). \tag{8}$$

Equations (5) through (8) might all be thought of being applicable to the one pass (no recirculation) trickling filter. The principal difference between equations (5) and (6) as compared to equations (7) and (8) seems to be in the interpretation of the organic load L. Phelps and Velz, for example, prefer the latter equations, and consider that L represents the removable organic load, rather than the applied organic load (2).

The distinction is made on the basis that some fraction of the applied load is untreatable.

By virtue of the simpler form, equations (7) and (8) have round wider application. Equation (8), for example, is well known as Velz's biological law for filter beds (3). Stack (4) has also used this principle in deriving an equation which makes use of the fractional removal for a unit depth.† Equation (7), involving time rather than depth, has its principal advocate in Howland, who has attempted to develop t in terms of the actual contact time of the waste in the bed (5). Under these circumstances K_t will vary considerably from the normal oxidation constant K_1 as noted by Phelps (6).

Schulze has made an interesting extension of Howland's analysis that time of contact is inversely proportional to the 2/3 power of the flow rate, I, and directly proportional to depth, D (7). Equation (7) then becomes

$$L_t/L_a = \exp(-K'_t \cdot D/I^{2/3}), \tag{9}$$

* For nomenclature, see Appendix E.

† Stack's equation reverts back to (8) when a simple mathematical adjustment is made. See Appendix A.

where $K_t' = K_t$ times the constant of proportionality in Howland's relationship.

At the present time, there does not seem to be enough experimental evidence to justify a strong preference between the parameters of time and depth represented by equations (7), (8), and (9). Furthermore, there is a distinct possibility that equations (5) and (6) have been unduly neglected since much raw data involves applied rather than removable loadings. As an example, the work reported by Howland in Fig. 2 of his article (5) seems to involve the type of deviation represented by equation (5).

In any case, equations (5) through (9) are known to have certain limitations. Temperature, for example, can influence purification rates. However, most authorities seem to agree that variation in K values due to temperature can be adjusted quantitatively by the van't Hoff–Arrhenius relationship.

Less clearly understood is the matter of whether under certain circumstances the equations might fail to hold completely. Velz (3) and Stack (4) both assume that there is a given loading above which the purification rate remains constant. That is to say dL/dt or dL/dD is a constant regardless of equations (5) through (8).

Velz and Phelps attempt to define the matter quantitatively by assuming that the limiting B.O.D. load is a function of the storage capacity of the bed which is in turn a function of medium surface area per unit volume (2).

Assuming that L_f, the B.O.D. load stored in the bed, is oxidized at a rate given by the first-order reaction equation, and that new B.O.D. is being added at the rate L_a in the filter of depth D_0

$$dL_f/dt = -K_1 L_f + L_a[1 - \exp(-K_D D_0)]. \tag{10}$$

At equilibrium $dL_f/dt = 0$, and the maximum load to be applied is

$$(L_a)_{max.} = K_1 L_f/[1 - \exp(-K_D D_0)]. \tag{11}$$

Some time can elapse before equilibrium is reached as may be noted integrating equation (10), which is of a linear, first-order type.

$$L_f = [L_a/K_1][1 - \exp(-K_D D_0)][1 - \exp(-K_1 t)] \tag{12}*$$

Stack introduces a somewhat different concept (4). His limiting load for L_a is given as that which will saturate one unit depth of filter with B.O.D. This quantity, designated S, is handled quantitatively by Stack in the following manner.

A coefficient of biosorption, j, is introduced and defined as the fractional removal per unit depth. Based on equation (8) with $D = 1$,

$$j = 1 - (L_D)_{D=1} = 1 - \exp(-K_D). \tag{13}$$

When a load S is applied to a unit depth, the maximum possible removal is jS. For a one pass filter with L_a greater than S, and X unit depths saturated, the quantity removal, L_r is given by

$$L_r = X_j S + (L_a - X_j S)[1 - (1 - j)^{(D_0 - X)}]. \tag{14}$$

This is based on the concept that the X saturated unit depths will remove the equivalent of the first term on the right-hand side of (14) while the unsaturated

* t in this equation represents the conventional time rather than time of contact.

VOL. 5 1961

$(D_0 - X)$ unit depths will remove organic material at the rate given by equation (8). (See Appendix A.)

Stack gives the loading that will saturate the filter as

$$L_s = S + j(D_0 - 1)S. \tag{15}$$

This is the minimum loading that would be necessary to insure a loading of S on the bottom unit depth.

Presumably with a loading L_s, the filter load L_f will be at its equilibrium point. L_s will then take on the equilibrium value of L_a in equation (11)

$$L_s = K_1 L_f / [(1 - \exp(-K_D D_0)]. \tag{16}$$

The maximum possible removal of B.O.D. then becomes

$$L_s[1 - \exp(-K_D D_0)] = K_1 L_f \tag{17}$$

This is also equal to $D_0 jS$ so that

$$S = K_1 L_f / jD_0 \tag{18}$$

Equation (18) represents the quantitative relationship between Velz's equilibrium load in the filter and Stack's loading to saturate a unit depth. The entire concept of a saturated filter is one which requires further experimental verification. If supportable, it would represent a powerful concept in trickling filter analysis.

Current thinking, for example, seems to be that within size ranges commonly used, type of media has no appreciable effect on filter efficiency (8). This is not in accordance with the Phelps–Velz theory which establishes the surface area per unit volume as an important criteria in filter capacity. Of course, the rate at which the organic load in the filter will oxidize is also of major importance. Adequacy of ventilation (and therefore bed porosity) presumably has an effect on the value of K_1.

In the event that filter saturation is to be considered, Stack's analysis seems somewhat more useful than Velz's, since the filter undoubtedly saturates at various depths in sequence. Equation (14) should be then a useful modification of Velz's law for a one pass trickling filter.

RECIRCULATION

The above comments have been directed principally towards the one pass filter. Recirculation of filter effluent at a rate R in addition to the raw inflow at rate I poses an additional problem in the analysis. The first attempt to quantitatively define the effect of recirculation was in the well known National Research Council Formula (9). The development of this equation introduced the use of series to trickling filter mathematics. These may usually be solved with the aid of the following two identities:

$$1/(1-x) = 1 + x + x^2 + x^3 + \ldots \tag{19}$$

$$1/(1-x)^2 = 1 + 2x + 3x^2 + 4x^3 + \ldots \tag{20}$$

In the National Research Council Analysis, a recirculation factor F was devised as a ratio by which the true volume of a filter could be multiplied to obtain the

effective volume. In the process a factor F' was derived which is the effective number of passes of water through the filter. The relationship is

$$F' = 1+R/I. \tag{21}$$

This does not take into account the fact that the availability of the organic matter decreases on each pass. Availability is used here not in the sense that the organic material is being removed by treatment, but rather that its treatability is decreasing. An availability factor f was introduced into the equation for F as follows

$$F = (1+R/I)/[1+(1-f)R/I]^2 \tag{22}$$

The derivation of both (21) and (22) is demonstrated in Appendix B.

When F is differentiated with respect to the recirculation ratio R/I (holding f constant) the result may be set equal to zero to find the value of R/I for maximum F. The following result obtains

$$R/I = (2f-1)/(1-f). \tag{23}$$

This implies an optimum value of recirculation ratio depending upon the value of f.*

Equations (21) through (23), while widely quoted, fail to relate recirculation and reaction kinetics. Velz (3) did this, assuming that in equation (8) D would be increased directly by the right-hand side of equation (21). Stack has given a more complete analysis, presenting the following equation for fractional removal when the applied load L_a is less than S.†

$$L_r/L_i = r(1+R/I)/(1+rR/I), \tag{24}$$

where

$$r = [1-\exp(-K_D D_O)]. \tag{25}$$

The expression L_r is the fractional removal of organic material figured on the basis L_i of the load present in the influent I. r is actual removal rate in the filter. Equation (24) is derived in Appendix C in a matter somewhat different from that presented by Stack.

Of great interest is the fact that this relationship may also be derived in the simple manner shown by Howland (5). Thus if Y_i is the B.O.D. in the influent, I, and Y_e is the B.O.D. in the filter effluent

$$L_r/L_i = 1-Y_e/Y_i, \tag{26}$$

and

$$r = 1-(I+R)Y_e/(IY_i+RY_e). \tag{27}$$

The solution of these equations for L_r/L_i in terms of r and R/I yields the Stack equation No. (24). The same conditions hold, that no unit depth of the filter may be saturated. The use of Howland's technique helps to clarify the relation between L_a and L_i, since equation (27) is simply another form of equation (8) or (25).

* f is given as about 0·9 (9).

† Stack made the requirement L_i less than S.

VOL. 5 1961

Equation (27) can therefore be rewritten as

$$r = L_r/L_a. \tag{28}$$

Combining equations (24) and (28) in the manner suggested by Stack (4),

$$L_a = L_i(1+R/I)/(1+rR/I). \tag{29}$$

Equation (29) is limited to the case where no unit depth is saturated. When X unit depths are saturated, the incoming loading L_i can be divided into two portions, XjS and (L_i-XjS). The applied load then becomes

$$L_a = XjS+(L_i-XjS)(1+R/I)/\left[1+(R/I)\,[1-(1-j)^{D_e-X}]\right]. \tag{30}$$

When this equation is combined with equation (13), the result is Stack's equation for a recirculating filter which is partially saturated.*

$$L_r = XjS+(L_i-XjS)(1+R/I)[1-(1-j)^{D_e-X}]/\left[1+(R/I)\,[1-(1-j)^{D_e-X}]\right]. \tag{31}$$

Neither equation (24) nor (31) take into account f, the N.R.C. availability factor. Now in Appendix B it is shown that f is used to modify the recirculation fraction $R/(I+R)$. If this procedure is followed explicitly in the development of equation (24) the following modification of (24) is obtained as may be seen in Appendix D.

$$L_r/L_i = r(1+R/I)/\left[1+R/I[1+f(r-1)]\right]. \tag{32}$$

However, if the recirculation factor is adjusted for f, that is, if it is altered to $fR/(1+fR)$, the result becomes

$$L_r/L_i = r(1+fR/I)/(1+rfR/I). \tag{33}$$

This is also demonstrated in Appendix D.

Now equation (33) can also be obtained in two more ways. If the factor f is considered to be a ratio applied to the treatment faction r on the second and all subsequent passes, equation (33) is easily obtained as may be demonstrated in Appendix D. Also, equation (33) may be developed by modifying equation (27):

$$r = 1-[(I+fR)Y_e/(IY_i+fRY_e)]. \tag{34}$$

When equation (34) is combined with equation (26), equation (33) is obtained.

The only interpretation of the physical aspect of f given in the literature is that by Fair and Geyer (1). These authors define f as the ratio of the fraction removed as given by equation (5) or (6) to that of (7) or (8), respectively. Thus in terms of K_D, for a given D_0:

$$f = 1-\{(1+K_DD_0n)^{-1/n}/[1-\exp^{(-K_DD_0)}]\}. \tag{35}$$

This concept of f seems to be more in accord with the assumptions used in obtaining equation (33) than (32). (Since equation (33) was derived by modifying r by a factor f exactly as in equation (35)). Equation (33) was also derived by altering the recirculation rate R rather than the recirculation ratio. When this is done in the N.R.C. derivation (See Appendix B) the result is the simple relationship

$$F = 1+fR/I. \tag{36}$$

* This equation appears as (13) in Stack's article (4).

In this equation f does not influence the maximum value of R/I as was the case with equation (23). Equation (36) says in effect that the lesser availability of the organic material is equivalent to that which would be accomplished by reducing the recirculation ratio.

In any case, f as defined by equations (35) and (36) may readily be introduced into Stack's equation for a partially saturated, recirculating filter. This involves the modification of equation (31) to the following form,

$$L_r = XjS + (L_i - XjS)(1+fR/I)[1-(1-j)^{D_s - X})]/\left[1+(fR/I)[1-(1-j)^{D_s-X}]\right]. \quad (37)$$

This equation, while cumbersome, represents no particular problem in the present era of high speed computers. Qualitatively it indicates the various factors which might influence the efficiency of a recirculating filter. Quantitatively its use is restricted by a lack of knowledge concerning certain of the variables such as j, S and f.

The equations in this section have dealt with depth as the parameter. As mentioned previously, Howland has utilized time (5). Corresponding to r for a one-pass filter there is defined a fractional removal p, where

$$p = [1-\exp(-K_t t_0)]. \quad (38)$$

This can also be written using Schulze's arrangement from equation (9)

$$p = [1-\exp(-K_t' D/I^{2/3})]. \quad (39)$$

When equation (38) or (39) is modified for recirculation the equivalence between p and r ceases. This is because the time of contact is decreased due to higher hydraulic loading. As noted previously, Howland has postulated that time of contact is inversely proportional to the 2/3 power of rate of application. The fractional removal for a recirculating filter then becomes

$$p_r = (1-\exp\{-K_t t_0[I/(I+R)^{2/3}]\} \quad (40)$$

This is easily converted into another form similar to the Stack equation (24)

$$L_r/L_i = p_r(1+R/I)/(1+p_r R/I). \quad (41)$$

The improvement in efficiency due to recirculation is apparently offset by the decrease in value of p_r compared to p, according to Schulze (7).

No provision is made for saturation in (41) and the basic relation for contact time in a one pass versus a recirculating filter is still subject to review. Bloodgood *et al.* have confirmed Howland's thesis (10), but Porges *et al.* have found that other parameters such as specific surface and kinematic viscosity must be accounted for (11).

SUMMARY AND CONCLUSIONS

In this paper the mathematic aspects of the trickling filter have been considered in some detail. In particular, this paper has been concerned with reaction kinetics and recirculation.

VOL. 5 1961

From the standpoint of reaction kinetics, it is shown that the removal of organic equivalent as measured by the B.O.D. test has been measured quantitatively by the first-order reaction rate or a modified first-order reaction rate which takes into account the degree of treatment achieved. The removal rate may be in terms of depth of filter or time of contact of waste with the bed.

Examples of the use of the first order reaction rate with depth of filter are Velz's and Stack's equations for a one-pass filter. (Stack's equation is shown, as might be anticipated, to reduce to the Velz equation with simple mathematical adjustment.) An example of the use of the first-order reaction rate with time of passage is the work of Howland and Schulze. Apparently the modified first-order reaction rate has not been widely used, although it might improve the data fit as for example in Howland's work.

Deviations from the accepted models for reaction rates are known to occur. Besides temperature, which can be handled quantitatively by the van't Hoff–Arrehnius equations, the major problem is associated with the saturation of the filter. Velz has postulated that the load contained in the filter is at equilibrium with incoming load when its rate of biological oxidation equals the incoming load. At this point, the entire filter may be said to be saturated. Stack prefers to work in terms of the loading which will saturate a unit depth, which may be a somewhat more useful concept. An equation is presented which attempts to relate Velz's equilibrium load and Stack's loading to saturate a unit depth.

The effect of recirculation is considered in terms of Stack's equations for a recirculating filter and the National Research Council's availability factor. Stack's equations, which are derived in a manner which permits more ready comparison with the availability factor, are based on first-order reaction kinetics. On the basis of the suggestion by Fair and Geyer that the availability factor represents the ratio of the removal by the modified to the true first-order reaction rate, Stack's equations are rederived to account for an availability factor. The results are not in agreement with those which are obtained when the availability factor is introduced in the manner by which it was originally derived. The conclusion is reached that the National Research Council availability factor does not have the relationship suggested by Fair and Geyer. In any case, it is shown that Stack's equations may be easily modified by a factor of the type suggested by Fair and Geyer.

The discussion on recirculation concludes with the presentation of Howland's equation for a recirculating filter. This is similar to Stack's except that saturation is not accounted for and removal is in terms of time of contract. Improvement in efficiency is offset by loss of contact time in this analysis. Howland's equation is based upon a non-linear relation with hydraulic loading rate. This has been somewhat confirmed, but there may be other important parameters which would influence contact time, according to Porges *et al.*

This review has demonstrated that the current theoretical analyses of trickling filter behavior stem from a somewhat common root. The current fluid state of trickling filter analysis suggests the need for a new approach to the behavior of

trickling filters. In this respect, the suggestion of Porges *et al.* that the analogy to a packed tower be utilized seems eminently worth while (11). In any case, the available theories would be well served by a carefully designed experimental pilot plant investigation. Among the topics which might profitably be examined are:

(a) Validity of first-order relationship.
(b) Time versus depth as a basis for B.O.D. extraction.
(c) The concept of filter saturation.
(d) Role of filter medium in filter saturation and time of contact.

These entities could probably best be studied in a pilot unit which was capable of close temperature control and which was fed artificially with a controlled, variable, organic stream which could be artificially diluted in a controlled manner.

Acknowledgement—A substantial portion of the work represented by this paper was accomplished while the author was engaged in the summer professorial program of the Engineering Command, U.S. Army Chemical Center, at Army Chemical Center, Maryland. The opportunity to approach this problem in such a fundamental manner was deeply appreciated.

APPENDIX A

In the nomenclature of his original article (4), Stack presents as equation (4) for a one-pass filter with a loading L less than S:

$$R = fL[1+(1-f)+(1-f)^2+\ldots+(1-f)^{D-1}].$$

The quantity in brackets may be recognized as a geometrical progression to D terms of 1 multiplied by the ratio $(1-f)$. The sum of such a progression is

$$C = (1)[1-(1-f)^D]/[1-(1-f)] = [1-(1-f)^D]/f.$$

Therefore

$$R = fL[1-(1-f)^D]/f \text{ or } R/L=(1-10^{-KD}),$$

which is exactly equivalent to (8) of this paper.

When X units are saturated, the fractional removal in the remaining $D-X$ units becomes by similar reasoning

$$[1-(1-j)^{D_e-X}].$$

This form may permit somewhat simpler calculations than that originally proposed by Stack.

APPENDIX B

At any given instant in a recirculating filter there is a fraction $I/(I+R)$ which is making its first pass. Now a fraction of a previous first pass fraction is on its second pass and so on. The average number of passes represented at any time is therefore a series which is given by

$$F' = 1 \cdot I/(I+R)+2 \cdot I/(I+R) \cdot [R/(I+R)]+3 \cdot I/(I+R) \cdot [R^2/(I+R)^2]+\ldots\ldots$$

$I/(I+R)$ is factored out and the remainder is seen to be of the type given by equation (20) with $x = R/(I+R)$.

A substitution is made in the left-hand side of equation (20) for x to simplify the series and the final result becomes

$$F' = (1+R/I).$$

When the N.R.C. availability factor, f, is introduced, it appears in concert with the fraction $R/(I+R)$. Thus the average effective passes of organic matter is

$$F=1 \cdot I \cdot f^0/(I+R)+2 \cdot I[f^1/(I+R)] \cdot R/(I+R)+3 \cdot I[f^2/(I+R)]R^2/(I+R)^2+\ldots$$

Once again equation (20) is used but now $x = fR/(I+R)$.
Following a procedure identical with that above

$$F = (1+R/I)/[1+(1-f)R/I]^2.$$

On the other hand, if f merely modifies R,

$$F = 1 \cdot I \cdot f^0/(I+fR)+2 \cdot I[f^1/(I+fR)]R/(I+fR)+3 \cdot I[f^2/(I+fR)]R^2/(I+fR)^2+.$$

and

$$F = (1+fR/I).$$

APPENDIX C

The fraction of the load removed in a filter of depth D_0 is given as r from equation (8)

$$r = (1-L_D/L_a) = [1-\exp(-K_D D_0)].$$

The following identities hold

L_1	=	removable B.O.D. entering first passage,
rL_1	=	B.O.D. removed on first passage,
$L_1(1-r)$	=	B.O.D. exiting first passage,
$L_1(1-r)[R/(I+R)]$	=	B.O.D. entering second passage,
$rL_1(1-r)[R/(I+R)]$	=	B.O.D. removed on second passage,
$L_1R(1-r)^2/(I+R)$	=	B.O.D. exiting second passage,
$L_1R^2(1-r)^2/(I+R)^2$	=	B.O.D. entering third passage,
$rL_1R^2(1-r)^2/(I+R)^2$	=	B.O.D. removed on third passage,

etc.
The total B.O.D. removed is L_r and is given by a series

$$L_r = rL_1+rL_1R(1-r)/(I+R)+rL_1R^2(1-r)^2/(I+R)^2+\ldots$$

rL_1 is factored from the above equation and the result is a series of the type of equation (19) with

$$x = (1-r)R/(I+R).$$

Substituting for x in the left-hand side of equation (19) to simplify the series,

$$L_r/L_1 = r(1+R/I)/(1+rR/I).$$

APPENDIX D

The factor f used in the National Research Council formula can be introduced into the recirculation formula in several ways. If applied exactly in the manner utilized in Appendix B, it will appear in concert with $R/(I+R)$.

Thus (modifying the development in Appendix C)

$$L_r = rL_i + r(1-r)fRL_i/(I+R) + r(1-r)^2f^2R^2L_i/(I+R)^2 + \ldots.$$

rL_i is factored out from the right-hand side and the remainder is a series of the type of equation (19) with $x = (1-r)fR/(I+R)$.

Simplifying,

$$L_r/L_i = r(1+R/I)/[1+R/I(1+f(r-1)].$$

A somewhat different result is obtained if the recirculated fraction is adjusted for f as follows $fR/(I+fR)$

$$L_r = rL_i + r(1-r)fRL_i/(I+fR) + r(1-r)^2f^2R^2L_i/(I+fR)^2 + \ldots$$

Now x in equation (19) is

$$x = (1-r)fR/(I+fR),$$

and the final result becomes

$$L_r/L_i = r(1+fR/I)/(1+rfR/I).$$

This latter result can be obtained by considering f as the factor by which the removal fraction r is reduced on the second and all subsequent passes.

VOL. 5 1961

The deviation is as follows:

L_i	=	removable B.O.D. entering first passage,
rL_i	=	B.O.D. removed on first passage,
$L_i(1-r)$	=	B.O.D. exiting first passage,
$L_i(1-r)R/(I+R)$	=	B.O.D. entering second passage,
$frL_i(1-r)R/(I+R)$	=	B.O.D. removed on second passage,
$L_iR(1-r)\ (1-fr)/(I+R)$	=	B.O.D. exiting second passage,
$L_iR^2(1-r)(1-fr)/(I+R)^2$	=	B.O.D. entering third passage,
$frL_iR^2(1-r)\ (1-fr)/(I+R)^2$	=	B.O.D. removed in third passage,

etc.

Therefore

$$L_r = rL_i + frL_i(1-r)R/(I+R) + frL_i(1-r)(1-fr)R^2/(I+R)^2 + frL_i(1-r)$$
$$(1-fr)^2R^3/(I+R)^3,$$

or $L_r = rL_i + frL(1-r)R/(I+R)[1+R(1-fr)/(I+R)+R^2(1-fr)^2/(I+R)^2+\ldots]$.

The series in brackets is equivalent to the left-hand side of equation (19) where $x = R(1-fr)/(I+R)$.

Therefore

$$L_r = rL_i + frL_i(1-r)R/(I+R) \cdot \left[1/[1-R(1-fr)/(I+R)]\right],$$

or

$$L_r/L_i = r(1+fR/I)/(1+rfR/I).$$

APPENDIX E

The following nomenclatures apply to all sections except Appendix A.

Nomenclature	*Description*	*Units*
D	Depth in filter	L
D_0	Overall depth of filter	L
d	Differential operator	None
e	Natural logarithm base	None
F	Effective passes of organic matter	None
F'	Effective passes of water	None
f	Availability factor	None
I	Influent rate (undiluted)	L^3T^{-1}
j	Fractional removal per unit depth	None
K_1	Oxidation coefficient	T^{-1}
K_D	Extraction coefficient	L^{-1}
K_t	Extraction coefficient	T^{-1}
K_t'	Modified extraction coefficient	$(T)^{-2/3}$
L	Organic load per unit area	$MT^{-1}L^{-2}$
L_a	Applied organic loading rate per unit area*	$MT^{-1}L^{-2}$
L_D	Loading remaining at any depth per unit area*	$MT^{-1}L^{-2}$
L_f	Load contained in filter per unit area*	ML^{-2}
L_i	Loading in undiluted influent per unit area* (equals L_a for one-pass filter only)	$MT^{-1}L^{-2}$
L_r	Loading removed per unit area*	$MT^{-1}L^{-2}$
L_s	Loading per unit area to saturate filter*	$MT^{-1}L^{-2}$
L_t	Loading remaining at any time per unit area*	$MT^{-1}L^{-2}$
n	Exponent expressing deviation from first order	None
p	Fractional removal by first order kinetics using time—one pass	None
p_r	Fractional removal by first order kinetics using time—recirculating	None
R	Rate of recirculation	L^3T^{-1}
R/I	Recirculation ratio	None
r	Fractional removal by first-order kinetics using depth as parameter	None
S	Loading per unit area which saturates one unit depth	$MT^{-1}L^{-3}$
t	Time of contact (except equation (12))	T
t_0	Time of passage from one pass filter	T
X	Number of unit depths	None
x	Variable quantity used in equations (18) and (19)	None
Y_i	B.O.D. in undiluted influent	M/L^3
Y_e	B.O.D. in effluent	M/L^3

* Organic load expressed as B.O.D. equivalent.

REFERENCES

1. Fair G. M. and Geyer J. C., *Water Supply and Waste-Water Disposal*. John Wiley, New York (1956).
2. Phelps E. B. (in collaboration with Velz C. J.), *Public Health Engineering* vol. 1, pt. 2. John Wiley, New York (1948).
3. Velz C. J., A basic law for the performance of the trickling filters. *Sewage Wks. J.* **20**, No. 4, 607–25 (1948).
4. Stack V. T., Jr., Theoretical performance of the trickling filter process. *Sewage Industr. Wastes* **29**, No. 9, 987–1001 (1957).
5. Howland W. E., Flow over porous media as in a trickling filter. *Proc. 12th Ind. Waste Conf. Purdue University*. Bull. No. 94, Purdue University Engineering Extension.
6. Phelps E. B., *Stream Sanitation*. John Wiley, New York (1944).
7. Schulze K. L., Hydraulic load, organic load, and efficiency in trickling filters. Presented at 32nd Annual Meeting, FSIWA, Dallas, Texas, October 1959, Department of Civil Engineering, Michigan State University, East Lansing, Michigan.
8. *Sewage Treatment Plant Design*. Manual of Practice No. 8, Federation of Sewage and Industrial Wastes Assns. (1959).
9. NRC Subcommittee Report, Sewage treatment at military installations. *Sewage Wks. J.* **18**, No. 5 (1946).
10. Bloodgood D. E., Teletzke G. H. and Pohland F. G., Fundamental hydraulic principles of trickling filters. *Sewage Industr. Wastes* **31**, No. 3, 243–53 (1959).
11. Sinkoff M. D., Porges R. and McDermott J. H., Mean residence time in a trickling filter. *J. Sanitary Engng. Div., Proc. A.S.C.E.* **85**, No SA6, paper 2251, 51–78 (1959).

VOL.
5
1961

KINETICS IN TRICKLING FILTERS

W. E. Howland, F. G. Pohland and D. E. Bloodgood

Purdue University

Abstract—Studies of the important effect on organic matter removal of the time of contact between synthetic sewage flowing over slime-covered spheres, flowing down plane surfaces variously inclined, and standing and surging while in contact with flat surfaces, are reported in this paper. In addition, a mathematical analysis of the problem of time of flow over small spheres holding capillary water is presented which seems to account for the results of recently reported laboratory studies. It is thought that this study adds support to the authors' approach to the fluid flow problem reported in earlier papers.

VOL. 5 1961

REMOVAL OF ORGANIC MATTER FROM SYNTHETIC SEWAGE FLOWING OVER SPHERES

McDermott (1) made a large number of runs to determine the rate of removal of organic matter from synthetic sewage flowing over a 23 ft high column of eighty-three separate $3\frac{1}{2}$ in. diameter rubber balls, one directly above the other. The rates of application varied between 18·5 and 272 mgad (million gallons per acre per day) figured on the basis of the projected area of one sphere. The removals were computed on the basis of the C.O.D. (chemical oxygen demand) tests run on samples which were initially filtered in order to eliminate the erratic effect of occasional sloughing of the slime. Each run, 10 min in duration, was made after natural sewage had been continuously recirculated over the balls to promote and maintain the slime growth. The synthetic sewage used was prepared from a mixture of one part by weight of sucrose, one of dextrose, one of soluble starch, and two of bactopeptone.

The time of flow (t) in seconds was computed by means of the empirical equation $t = 30{\cdot}2\ D^{1{\cdot}08}\ Q^{\ 0{\cdot}515}$ where D is the depth of the filter in feet and Q is the rate of application of sewage on the top ball in mgad, based upon data of Gutierrez (2). This is a preliminary form of the equations which are presented in reference (6). Time had been computed by dividing the weight of liquid on the balls during the run by the measured rate of flow.

The temperature was not controlled in these experiments since the balls were exposed to the changing conditions of convection and other currents of air in two stories of an open barn-like laboratory. To these disturbing variations and to the observed varying conditions of the slime may be ascribed some of the variation of the K factors which are shown in Table 1. (K is the numerical part of the exponent of 10 in these equations.) The constants appearing in these equations were computed by the methods of least squares on the assumption that the curves plot as straight lines on semi log paper.

TABLE 1. SUMMARY OF THE SEVEN EMPIRICAL EQUATIONS OBTAINED FOR SPECIFIC INITIAL CONCENTRATION RANGES

(Normal slime conditions)

Initial concentration of synthetic sewage applied. Range of C.O.D. (ppm)	Formulas
178–236	$p = 0{\cdot}985 \times 10^{-0{\cdot}00051t}$
395–430	$p = 0{\cdot}940 \times 10^{-0{\cdot}00098t}$
585–674	$p = 1{\cdot}034 \times 10^{-0{\cdot}00090t}$
714–760	$p = 1{\cdot}031 \times 10^{-0{\cdot}00462t}$
795–805	$p = 0{\cdot}985 \times 10^{-0{\cdot}00358t}$
930–980	$p = 1{\cdot}022 \times 10^{-0{\cdot}00004t}$
1000–1020	$p = 0{\cdot}972 \times 10^{-0{\cdot}00465t}$
General equation	$p = 1{\cdot}012 \times 10^{-0{\cdot}00112t}$

(Here p is the fraction of the original organic matter remaining after time t, in seconds, of contact had elapsed.)

Another somewhat similar series of experiments was made earlier by Gutierrez (2), employing methods similar to those described above, but using only twenty one balls, which were $3\frac{1}{2}$ in. in diameter in a column approximately 6 ft high. The rates of application varied from 112 to 448 mgad (or from 7·5 to 30 ml/sec on the top ball). The time of contact between the balls and the sewage was extended by running the synthetic sewage over the balls as many as five times (or five passes).

Gutierrez (2) summarized his results in the form of three equations for three separate rates of application or three separate calculated times of a single pass. In these equations y is the ppm of B.O.D. as calculated from the C.O.D. in the effluent and x is the number of passes 1, 2, 3, 4 or 5. When the time of a single pass was 18·5 sec, the equation became $y = 231 \times 10^{-0{\cdot}17x}$; when it was 12·5 sec, the equation became $y = 181 \times 10^{-0{\cdot}14x}$; and when it was 8·6 sec, the equation became $y = 93 \times 10^{-0{\cdot}09x}$.

If the general form of the equation adopted by McDermott in Table 1 is assumed, the exponents may be equated and $0{\cdot}17x$ may be equated to K 18·5, $0{\cdot}14x$ to K 12·5 and $0{\cdot}09x$ to K 8·6 and x may be taken as 1. The resulting values of K, namely 0·0092, 0·0112 and 0·0105, average, say, 0·01, may then be taken as representing the results of this work. That it is higher than McDermott's value of 0·004, may be due to the fact that the operation of collecting and passing the sewage over the balls successively five times provided additional time of contact between the sewage and the slime floating in the fluid. Also additional time is required to drain the balls between passes.

In his work Rempe (3), using synthetic sewage and methods similar to those of McDermott and Gutierrez, measured organic removals on inclined plane surfaces varying in slope from 0·05 to 1·00—the tangent of the angle of the plane with the horizontal. The empirical equations chosen to represent these data are shown in Table 2.

TABLE 2. EQUATION OBTAINED FROM TESTS USING SYNTHETIC SEWAGE
(Sewage passed over the Rempe apparatus more than once.)

Run No.	Initial influent concentration of C.O.D. (ppm)	Equation
1	105	$e = 93{\cdot}6\times10^{-0.001444t}$
2	145	$e = 90{\cdot}6\times10^{-0.001950t}$
3	269	$e = 250\times10^{-0.001291t}$
4	329	$e = 320\times10^{-0.00078t}$
5	442	$e = 335\times10^{-0.001075t}$
6	626	$e = 587\times10^{-0.00092t}$
7	1245	$e = 1107\times10^{-0.00065t}$
8	236	$e = 213\times10^{-0.001257t}$
9	236	$e = 210\times10^{-0.001265t}$
10	236	$e = 187\times10^{-0.001174t}$
11	238	$e = 227\times10^{-0.001658t}$
12	238	$e = 200\times10^{-0.001705t}$
13	240	$e = 210\times10^{-0.001990t}$
14	234	$e = 209\times10^{-0.001547t}$
15	208	$e = 161\times10^{-0.000897t}$
16	208	$e = 181\times10^{-0.000907t}$
17	208	$e = 167\times10^{-0.001033t}$
Combined data from runs 8 through 14 inclusive	238 ± 4	$e = 207\times10^{-0.00146t}$
Combined data from runs 15 through 17 inclusive	208	$e = 164\times10^{-0.001257t}$

e = ppm C.O.D. in effluent.
t = time of contact in seconds.

The results of a typical run are shown in Fig. 1. In these experiments the time of contact between the sewage and the slime ranged between 330 to 500 sec, while the time of flow for a single pass varied from 23 to 94 sec. The time was extended by passing the sewage over the slime several times as in the experiments by Gutierrez. Attention is called to the fact that although a straight line was adopted to represent the data, the data themselves seem to form a curved line which becomes flatter as time increases. But it appears likely that if B.O.D. rather than C.O.D. had been used as the measure of organic content that this line would have been more nearly straight since experiments indicate that the ratio of B.O.D. to C.O.D. content of treated synthetic sewage appears to decrease as the treatment continues. It is noted that this same decreasing rate of purification or decreasing K value has been observed by others as indicated by Howland (5) in reporting on the operation of the Baltimore, Md. trickling filters.

Analysis was made of a series of tests by Rempe (3) in which the synthetic sewage was passed down an incline only once and in which the time of contact varied from 23 to 94 sec. The equation representing this data is $e = 207\times10^{-0.00405t}$. The higher

VOL. 5 1961

value of K, namely 0·00405 sec^{-1}, indicates also the higher rate of removal that occurs in the early stage of the purification process.

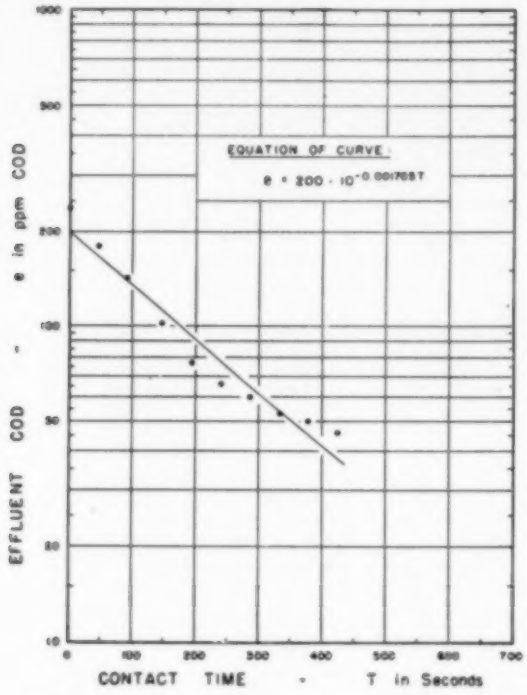


FIG. 1. Effect of time on C.O.D. remaining in sewage down an inclined slope.

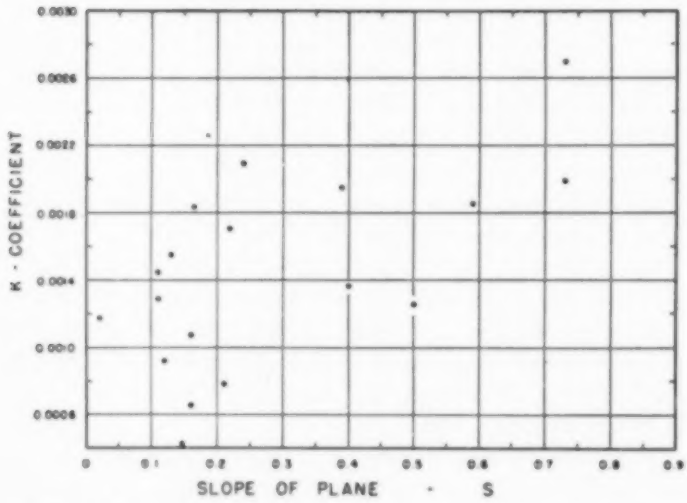


FIG. 2. Effect of slope of inclined plane on rate of removal of organic matter as measured by K.

Comparing Rempe's results on inclined planes with those of Gutierrez and McDermott on balls, one may conclude that for comparable times the balls do a better job, i.e. the K values are higher. The reason why the balls seem to give better results are several: at comparable rates of flow the velocities are higher; the liquid films are thinner; and at the bottom of each ball, there would appear to be a chance for surface layers to exchange places with the lower ones. All of these effects tend to provide more contact between the various parts of the liquid and the slime.

To explore the relationship of these factors of liquid depth and velocity, it would be helpful to know what relation there may be between the slope of the incline and the resulting K values. Fig. 2 shows a plotting of K against the tangent of the slope angle, but, although there may be some indication of an expected positive correlation, certainly none is proved—the data vary too much.

The thesis work of Walters (4) throws some light on the effect of velocity upon K. Using synthetic sewage and methods described previously, he investigated the effect of static contact between the sewage and slime on a flat horizontal surface and found that the rate of removal that occurs in the first minute of contact corresponds to a K value of 0·0037 sec^{-1} i.e. similar to the K value Rempe found for the corresponding initial time on inclined surfaces. The data further indicated that increasing the time of contact causes very little additional purification so that the overall K values continually decrease as shown in Table 3.

VOL. 5 1961

Table 3. Decreasing K values with time

Time of contact (min)	Percentage C.O.D. remaining	Calculated K value (sec^{-1})
1	60·2	0·00370
2	64·0	0·00160
3	68·0	0·00093
4	60·6	0·00090
5	67·5	0·00059
10	76·0	0·00020

In making these tests, Walters chose a depth of sewage in the apparatus of 0·018 in. to be comparable with the thickness used in Rempe's experiments which covered the approximate range of from 0·007 to 0·043 in. In McDermott's experiments the approximate range of thickness was from 0·0047 to 0·017 in.

Walters also found that the agitation caused by tipping the apparatus back and forth as frequently as 5 tips per min had very little effect in accelerating the purification process, that is to say, had little effect on K.

Another interesting finding on the effect of intermittent application of sewage to the slime surface was made by Rempe. Using an inclined plane at a slope of 0·21, a contact time of 26·8 sec, and sewage with a concentration of 322 ppm C.O.D.,

with the sewage applied continuously for 15 min, samples of the effluent were taken at 5 min intervals. The flow was then stopped for 15 min giving the slime a chance to oxidize the organic matter it had adsorbed or, at any rate, to restore the purify-

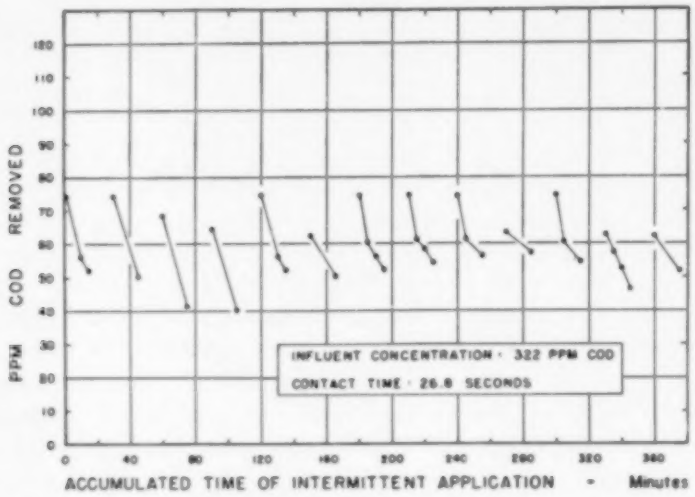

PPM COD REMOVED
INFLUENT CONCENTRATION - 322 PPM COD
CONTACT TIME - 26.8 SECONDS
ACCUMULATED TIME OF INTERMITTENT APPLICATION - Minutes

FIG. 3. Effect of intermittent application on C.O.D. removal on inclined planes.

ing powers. This cycle of operation was repeated several times and the results are shown in Fig. 3.

Conclusions in regard to "contact time"

These studies show that time of contact between sewage and slime is an important factor in determining the removal of organic matter by a trickling filter.

Additional studies are warranted to develop suitable methods for increasing the "contact time" in a trickling filter such as by using slabs with flat slopes as media.

FURTHER STUDIES ON THE RELATION BETWEEN TIME OF FLOW AND RATE OF FLOW THROUGH A BED OF SPHERES

Since the publication of the two papers on this subject from Purdue (5), (6), an interesting paper has appeared describing the experiments of Sinkoff *et al.* (7) at the Taft Laboratory of the United States Public Health Service in Cincinnati, Ohio, which at first reading might seem to cast doubt on the value of the approach to the problem presented by the authors in the earlier papers mentioned. In this paper the attempt will be made to show that the new data lend support to the concept of the earlier papers and, in addition, reveal the importance of a new phenomenon associated with small spheres not previously considered, namely, capillary water at the points of contact between the spheres.

To determine the amount of capillary water held between two spheres it is helpful to know the mathematical relation between the amount of this water and the radius, a (or diameter $2a$), of the circle through the meniscus shown on Fig. 4, since this

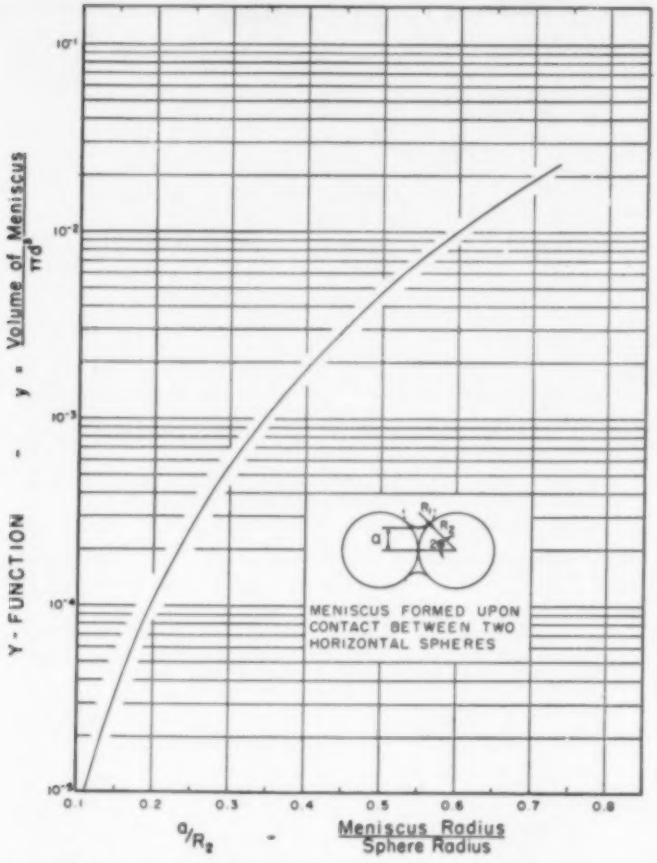


FIG. 4. Relation of diameter of the meniscus trough to the volume of capillary water between spheres.

distance is the most easily measured dimension. This relation is shown by the curve of Fig. 4. The ordinate to this curve is the expression in the bracket of the following formula for the volume of the fluid within the meniscus:

$$V = \pi(d_2)^3 \frac{\sin^4\Theta}{\cos^2 2\Theta}\left[1 - \tan 2\Theta\left(\frac{\pi}{2} - 2\Theta\right)\right] \tag{1}$$

Here $d_2 = 2R_2$ as shown in Fig. 4, where R_2 = radius of sphere.

The abscissa is given by the equation:

$$\frac{a}{R_2} = \frac{\sin 2\Theta + \cos 2\Theta - 1}{\cos 2\Theta}. \tag{2}$$

VOL. 5 1961

TABLE 4. GEOMETRICAL PROPERTIES OF MENISCUS BETWEEN SPHERES

Θ (degrees)	$\frac{V}{\pi d_1^3}$ (ordinate)	a/R_2 (abscissa)
$3^1/_2$	0·00001192	0·1153
4	0·00001930	0·1307
5	0·00004495	0·1609
$7^1/_2$	0·0002009	0·2326
10	0·0005723	0·2998
$12^1/_2$	0·001258	0·3629
15	0·002365	0·4226
20	0·006259	0·5337
25	0·01303	0·6362
30	0·02355	0·7320

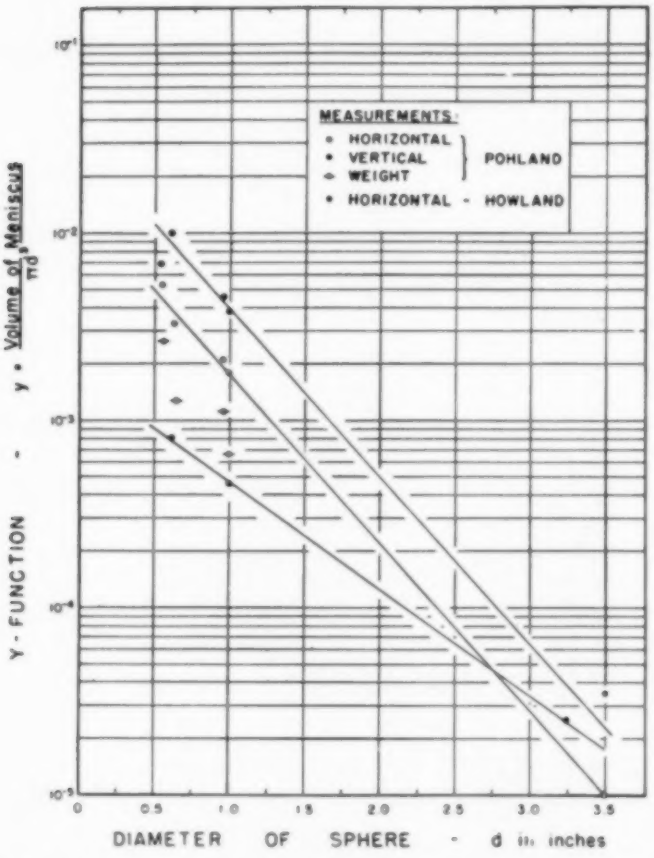


FIG. 5. Effect of diameter of spheres on volume of capillary water between spheres.

Calculated values from which the curve in Fig. 4 is plotted are shown in Table 4.

Using measurements made by Howland of the diameter of the circle through the trough of the meniscus here called $2a$, between two horizontal spheres side by side, each 0·6 in. in diameter, then between two spheres similarly placed each 1 in. in diameter, and finally between liquor glasses formed in the shape of $3\frac{1}{4}$ in. hemispheres similarly placed, and also using the curve of Fig. 4, a relation was established between the diameter of the sphere and the volume of the capillary water held within the meniscus. The adopted relation is the lowest curve of Fig. 5. Other measurements made subsequently are shown also on Fig. 5, all of them giving higher values of this volume than were first obtained by Howland. Since Howland's data give the least value of the volume of capillary water, his values were used in subsequent calculations. For the purpose of this analysis it is necessary only to show thaf there is enough capillary water storage to account for the observed time delay. It there is more than enough, one may assume that all of the water so stored does not participate in the delaying action.

It is here proposed that what Sinkoff *et al.* called "residence time" is the sum of the time of flow around the spheres computed by Sinkoff's equation,

$$\frac{t_r}{H} = 0{\cdot}0136\left(\frac{S}{Q}\right)^{2/3},$$

VOL. 5 1961

which is a form of Howland's equation as Sinkoff (7) shows, plus the time for the particulate or dissolved material like the sodium chloride in his experiments to pass through the water held by capillarity, namely the volume of the capillary water divided by the corresponding rate of flow. Combining the expression for the two components of "residence time", the equation becomes:

$$\frac{t_r}{H} = 0{\cdot}0136\left(\frac{S}{Q}\right)^{2/3} + \frac{ndy}{2}\left(\frac{S}{Q}\right) \qquad (3) \text{ or}$$

$$\frac{t_r}{H} = \frac{1{\cdot}3(3\nu)^{0{\cdot}33}}{g^{0{\cdot}33}}\left(\frac{S}{Q}\right)^{0{\cdot}67} + \frac{ndy}{2}\left(\frac{S}{Q}\right). \qquad (3)$$

Here t_0 is residence timed in seconds, H is depth of filter in feet, S is π/d the "specific surface" as assumed by Sinkoff, d is diameter of sphere in feet, Q is rate of flow in cubic feet per second per square foot of filter, y is the meniscus volume function (volume of meniscus $+ \pi d^3$, ordinate of Fig. 4 and 5), n is the number of meniscus contacts per sphere, ν is viscosity $= 1{\cdot}23 \times 10^{-5}$ ft²/sec, and $g = 32{\cdot}2$ ft/sec².

Sinkoff *et al.* have shown what the porosity actually was for each of their experiments. Their 3 in. spheres were so loosely packed (porosity = 0·48) as to correspond with cubical packing in which the number of contacts is 6; the $\frac{1}{2}$ in. and 1 in. sphere filters had a porosity of from 0·41 to 0·42, corresponding to spheres arranged in an hexagonal pattern in each layer, but one resting directly above the other in which arrangement there are eight contacts; the $\frac{3}{4}$ in. spheres were packed more closely (porosity = 0·37) but not nearly so close as to give the most dense packing which

would have yielded twelve contacts. The number 9 was more or less arbitrarily assigned to the case where the porosity was 0·37. The porosity associated with each of the different types of packing ranges from 25·95 to 47·64 per cent (8).

The calculation of values of t_r/H from equation (3) is shown in Table 5 and the values are plotted as curves C_1, C_2, C_3, and C_4 on Fig. 6. On Fig. 6 are shown the two empirical curves of Sinkoff *et al.*, *A* and *B* as well as Howland's original curve, C. The agreement between Sinkoff's curve *A* for the smallest spheres and the new theory herein presented seems very good.

It is to be noted that the values 0·48, 0·41 and 0·42 from Sinkoff's experiments have been interpreted as porosities. If they were in fact void ratios, then the packing was closer than has been assumed herein, and there were more contacts and more corresponding capillary water than has been estimated — more than enough to account for the time delay here calculated. As stated above, it seems plausible that not all of the capillary water participates in the delaying action.

The disagreement between the theoretical curve *C* and empirical curve *B* for the largest spheres will now be discussed. There are two possible reasons why in the empirical equation: $t =$ a constant divided by Q^n, n is less than 2/3 for the largest balls of the experiment cited. (Here t is the time of flow through a given filter with a given size of sphere, Q is the rate of flow through the filter). Instead of sheet flow, as assumed in the derivation of the 2/3 law, there was, as mentioned by Sinkoff *et al.* on the large sphere, a breaking up of the sheet into small rivulets when the flow was small. Rivulets were observed in several of the Purdue experiments cited. (In some of the experiments a detergent was added to prevent this effect.) As the flow on the spheres increased the rivulets undoubtedly widened out into broader sheets. The effect was to cause the velocity to remain more nearly constant as the flow increased than if, at low flows, the rivulets did not exist, but, instead, there was a continuous sheet at all times. In the experiments on small spheres of Sinkoff *et al.* the flows were larger per unit horizontal surface of filter than in their experiments on large spheres. There was undoubtedly also a better distribution from sphere to sphere when the diameters were small.

When the spheres were large the liquid introduced at the top of the filter probably did not immediately spread all over the surface of the filter but flowed directly downward through the very large openings or channels between the spheres which open up like funnels to receive the flow. In the upper part of the filter, probably most of the flow occurred on the spheres immediately below the edge of the distributing cone of the Sinkoff apparatus. When the spheres were small the obstruction to flow offered by the spheres of small dimensions caused the flow to spread laterally near the surface so that a few inches below the surface the flow appeared to be quite uniformally distributed from sphere to sphere as noted. Furthermore with small spheres the flows were larger. Thus when the spheres were large the flow was channeled and not well distributed in the upper part of the filter. Also the velocity was higher and the flow was more likely to be turbulent over a larger fraction of the filter depth than in the filter with small spheres.

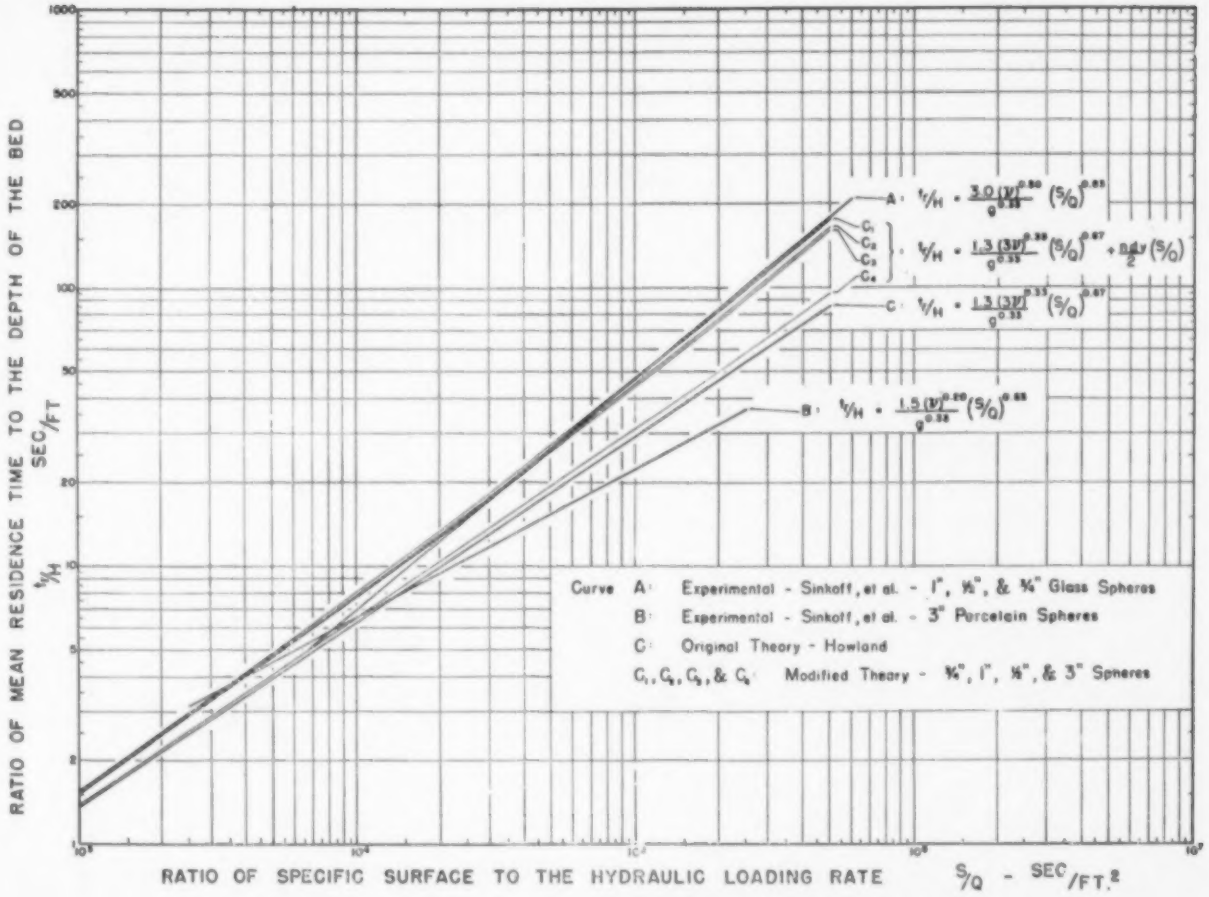

RATIO OF MEAN RESIDENCE TIME TO THE DEPTH OF THE BED
t/H SEC/FT
RATIO OF SPECIFIC SURFACE TO THE HYDRAULIC LOADING RATE S/Q - SEC/FT.²
A: t/H = 3.0(V)^0.50/g^0.33 (S/Q)^0.83
C1 C2 C3 C4
t/H = 1.3(3V)^0.33/g^0.33 (S/Q)^0.67 + 0.47/2 (S/Q)
C: t/H = 1.3(3V)^0.33/g^0.33 (S/Q)^0.67
B: t/H = 1.5(V)^0.50/g^0.33 (S/Q)^0.83
Curve A: Experimental - Sinkoff, et al. - 1", ½", & ¼" Glass Spheres
B: Experimental - Sinkoff, et al. - 3" Porcelain Spheres
C: Original Theory - Howland
C1, C2, C3, & C4 Modified Theory - ¼", 1", ½", & 3" Spheres

FIG. 6. Effect of capillary water on time of flow through a bed of spheres.

It can be shown that when the flow is turbulent rather than laminar the time of flow through a given depth of filter varies inversely with the 1/3 power of the rate of flow rather than with the 2/3 power. Thus, because of the greater extent of

TABLE 5. COMPUTATION OF POINTS ON CORRECTED TIME CURVE FOR FLOW OVER SPHERES

d (in.)	d (ft)	y from Fig. 5	n	ndy / 2
1/2	1/24	0·0009	8	0·00015
3/4	1/16	0·00065	9	0·00018
1	1/12	0·00047	8	0·00016
3	1/4	0·000024	6	0·00002

S/Q	$(S/Q)^{2/3}$	$0·0136\,(S/Q)^{2/3}$	$0·00015\,S/Q$	$0·00018\,S/Q$	$0·00016\,S/Q$	$0·00002\,S/Q$
1000	100	1·36	0·15	0·18	0·16	0·02
5000	291	3·96	0·75	0·90	0·8	0·1
10,000	464	6·26	1·5	1·8	1·6	0·2
50,000	1356	18·4	7·5	9	8	1
100,000	2150	29·3	15	18	16	2
500,000	6280	85·5	75	90	80	10

Values of t_r/H from equation (3)

S/Q	Diameter (in.)			
	0·5	0·75	1	3
1000	1·51	1·54	1·52	1·38
5000	4·71	4·86	4·76	4·06
10,000	7·76	8·06	7·86	6·46
50,000	25·9	27·4	26·4	19·4
100,000	44·3	47·3	45·3	31·2
500,000	160·5	175·5	165·5	95·5

the turbulent region in the filter with the large spheres, n tended to depart farther from the value of 2/3 than in the filter with small spheres.

To show that the value of n is 1/3 for turbulent sheet flow one has only to assume that the Chezy formula applies, that is, that $V = C\sqrt{(rs)}$ where V is velocity, C is a constant, s, the slope, is also a constant, and r is the hydraulic radius which in the case of sheet flow is substantially the same as thickness of liquid film. Thus velocity varies with the 1/2 power of the thickness. The flow varies with the area of cross-section of flow, that is to say, with the thickness, as well as with the velocity which, as shown, varies with the 1/2 power of the thickness; hence flow varies with the 3/2 power of the thickness or thickness varies with the 2/3 power of the flow. Now since velocity varies with the 1/2 power of the thickness, the velocity varies with the $1/2 \times 2/3 = 1/3$ power of the flow and time inversely with the 1/3 power of the flow.

THEORY OF FLOW DOWN THE INSIDE OF A PARTIALLY FILLED CYLINDER

Another study was made to determine whether or not the steeper slope of the curve for the smaller spheres (curve *A* of Fig. 6) might not be due to the behavior of the channels between the spheres acting like partially filled vertical cylindrical tubes. To do this, equation (4) was derived:

$$Q_1 = \frac{\pi w r_2^4}{8\mu}\{1-4\beta^2+3\beta^4-4\beta^4 \ln \beta\}. \tag{4}$$

Here Q_1 is rate of flow inside of the cylinder, w is unit weight of water in pounds per cubic foot, r_2 is radius of cylinder in feet, μ is the viscosity in terms of pound seconds per square foot, and β is the ratio of the radius of the hollow center inside of the flowing fluid (r_1) to the inside radius of the cylinder (r_2) as shown on Fig. 7.

The volume of liquid in a cylinder 1 ft high would be

$$V = \pi r_2^2(1-\beta^2), \tag{5}$$

so the average time of travel, T, per foot of height of tube, L, would be V divided by Q_1, or

$$T = \frac{V}{Q_1} = \frac{8\mu}{w r_2^2}\left\{\frac{1-\beta^2}{1-4\beta^2+3\beta^4-4\beta^4 \ln \beta}\right\}. \tag{6}$$

Then,

$$\frac{T w r_2^2}{8\mu}$$

is equivalent to the expression in the bracket of equation (6) and has been plotted as the ordinate of Fig. 7, and

$$\frac{\pi w r_2^4}{8\mu} \cdot \frac{1}{Q_1}$$

is equivalent to the inverse of the bracket in equation (4) and has been plotted as the abcissa of Fig. 7.

A calculating machine was used for computing these co-ordinates in order to be able to determine with precision the slope of the various parts of the curve. These slopes are shown in a table on Fig. 7. The values of β for the plotted points are shown on the curve. The slopes indicated were computed from the calculated values of the co-ordinates of adjacent pairs of these points.

From the study of this curve it is evident that the slope at the left is 3/4 and that it gradually changes to a slope of 2/3 at the right hand side. Hence it appears that when there is only a thin film of fluid flowing down the inside of a cylindrical tube the slope of the curve is 2/3 as expected, but that as the tube fills up, the slope becomes more and more nearly equal to 3/4. As soon as the tube is filled, i.e. when β equals zero, further increases in flow would cause heading up on the top of the tube and the slope would suddenly change to unity.

VOL. 5 1961

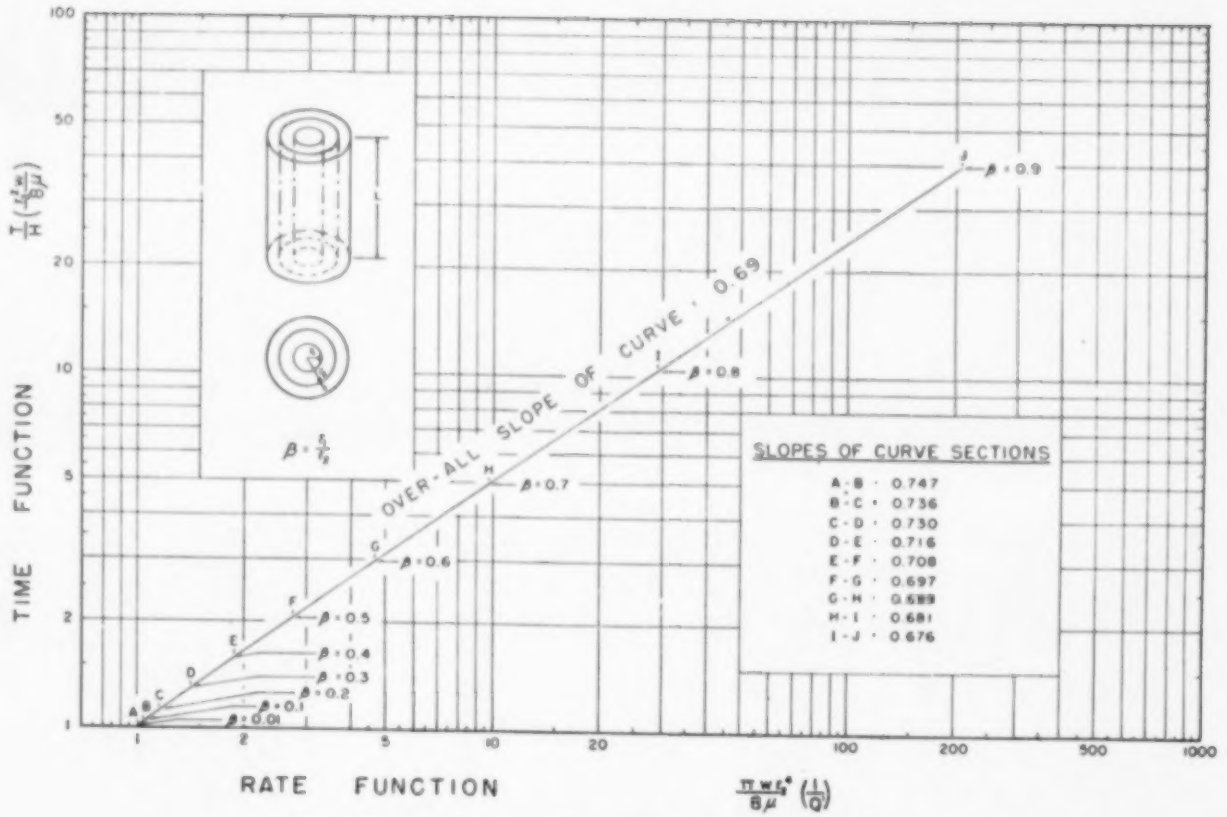


FIG. 7. Time of flow through a partially filled vertical cylinder.

The question is, can the greatest slope of the Sinkoff curves for small spheres be explained by this effect? Or in other words, does the Sinkoff data fall on the part of this curve where the slope is near to 3/4? To answer this question one may assume the highest flows in the Sinkoff experiments which occurred with the runs on the smallest spheres at the lowest value of S/Q which will be taken as 8000. Since

$$S = \pi/d = 24\pi,$$

$$Q = \frac{24\pi}{8000} \cdot \frac{\text{ft}^3}{\text{ft}^2\ \text{sec}};\quad t_r/H = 0{\cdot}0136(8000)^{2/3} = 5{\cdot}65\ \text{sec/ft}.$$

The volume of fluid in ft³ of filter is

$$Qt_r/H = \frac{24\pi}{8000} \text{ times } 5{\cdot}68 = 0{\cdot}0536.$$

The porosity in this case was reported to be 0·41, hence 0·0536 + 0·41 = 0·131 of the space between the spheres was filled with liquid. This corresponds to a value of $\beta = r_1/r_2 = \sqrt{1-0{\cdot}131} = 0{\cdot}93$. If the void ratio and not the porosity was 0·41 in this case, then porosity 0·29 and 0·0536 + 0·29 = 0·185 of the space between the spheres was filled with liquid and $\beta = r_1/r_2 = \sqrt{1-0{\cdot}185} = 0{\cdot}903$. As may be seen from Fig. 7 these values of β are in the region of the curve where the slope is very close to 2/3.

Thus it would appear that one must fall back upon the concept of capillary water in order to explain the slope of the Sinkoff curves for small spheres.

Equation (4) is thought to be correct for the following reasons. When $\beta = 1$ it yields the well-known equation for vertical flow down a flat plate whose width is $2\pi r_2$ as derived (5). When β equals zero the equation becomes the well-known Poiseuille equation.

Conclusions in regard to theory of fluid flow through trickling filters

When the media is small the effect on "residence time" of capillary water may be as large as 100 per cent when the rate of liquid application is low.

On stone surfaces the flow is thought to be well distributed by the rotating arms employed. Consequently the channeling which causes n in the formula ($t =$ a constant $\div Q^n$) to be less than 2/3 may not occur in sewage filters. The stones in trickling filters more nearly resemble the large spheres in the Sinkoff experiments than the small ones, consequently the capillary effect which causes n to be larger than 2/3 is likely to be very small. Such capillarity as exists will, it is thought, tend to compensate—in its effect on n—for the effect of channeling. At usual rates of dosing, the spaces between the stones probably will not be filled or rather will not even come close to the range of nearly filled cylindrical tubes where the value of n approaches 3/4 but, instead, will be in the region where n is close to 2/3. For these reasons it is believed that a value of $n = 2/3$ may be about right for the usual sewage trickling filter. To settle this matter careful studies on actual operating trickling filters are needed.

VOL. 5 1961

REFERENCES

1. McDermott, J. H., Influence of media surface area upon the performance of an experimental trickling filter (1957).*
2. Gutierrez, L. V., Jr., The hydraulics and organic removal capacity of an experimental trickling filter (1956).
3. Rempe, J. E., Jr., Influence of contact time upon purification capacity (1957).
4. Walters, C. F., The effect of contact time obtained by static detention, on the purification by a biological slime (1959).
5. Howland, W. E., Flow over porous media as in a trickling filter. *Proc.* **12***th Ind. Waste Conf. Purdue University*, September 1958, vol. XLII, No. 3, p. 435.
6. Bloodgood, D. E., Teletzke, G. H. and Pohland, F. G., Fundamental hydraulic principles of trickling filters. *Sewage Industr. Wastes* **31**, 3, p. 243 (1959).
7. Sinkoff, M. D., Porges, R. and Mc Dermott, J. H., Mean residence time of a liquid in a trickling filter. *J. Sanit. Engng Div. of A.S.C.E.* **85**, No. SA6, p. 51 (1959).
8. Dallavalle, J. M., *Micromeretics* p. 105. Pitman (1943).

DISCUSSION

W. W. Eckenfelder: There seems to be little question that biological filtration data can be related to depth and hydraulic loading considered as a function of contact time. However, just as in the activated sludge process, B.O.D. removal efficiency depends upon both time and activated sludge solids, it would seem reasonable that B.O.D. removal efficiency in trickling filtration will depend upon both contact time and the mean active film surface per unit depth. Since the mean film surface per unit depth has been observed to vary with depth, temperature and other factors, modification to the author's formulation would seem warranted in some cases In addition, as Howland has observed, removal does not always follow first-order kinetics. but usually more closely approximates a retardant type function. In the evaluation of various published filtration data I have found the following retardant type function to closely describe filtration performance:

$$\%_{\text{remaining}}\ \text{B.O.D.} = \frac{1}{(1+C^m/Q^n)}.$$

In the above equation the exponents n and m were found to vary from 0·365 to 1·49 and 0·175 to 1·129, respectively.

* This study, like references (2) (3) and (4) is a thesis written for the Master's Degree in Civil Engineering at Purdue University; it is on file in the library at this school.

ELEMENTS OF A TRICKLING FILTER THEORY

Dr. K. L. SCHULZE

Michigan State University

USING an experimental trickling filter which has been described in two previous publications (1, 2) it was shown that the efficiency measured as the fraction of B.O.D. remaining was constant up to 11 lb of B.O.D. per yd³ per day as long as the hydraulic load remained constant. Changing the hydraulic load showed that the fraction of B.O.D. remaining was proportional to the 2/3 power of the hydraulic load. According to Howland (3) and Bloodgood *et al.* (4), the contact time for trickling filters is directly proportional to the depth of the filter, D, and inversely proportional to the 2/3 power of the hydraulic load Q. Assuming that the fraction of B.O.D. remaining is directly related to contact time the efficiency can be expressed by the equation:

$$L_e/L_i = 10^{-KD/Q^{0.66}} \tag{1}$$

where L_e = final effluent B.O.D. (mg/l.),
L_i = initial B.O.D. (mg/l.),
D = depth of filter (ft),
Q = hydraulic load (mgad),
K = constant.

If the value of K is known, this equation will predict the efficiency of single pass filters for any combination of depth and hydraulic load as long as the critical limit of organic load has not been reached. Plotting of data showed that $K = 0{\cdot}3$ for the experimental screen filter. Nearly the same K value was found for data from British filters using conventional stone media (5). Equation (1) is similar to the one used by Velz (6), except that the Velz equation does not include hydraulic load as a factor. Velz's constant K_1 is therefore equal to $K/Q^{0.66}$.

Besides depth and hydraulic load a temperature correction factor b can be introduced into equation (1) so that:

$$L/L_i = 10^{-b K_{20}{}^{D/Q^{0.66}}}, \tag{2}$$

where $b = 1{\cdot}035^{T-20}$ according to Howland (3). A critical survey of data available so far indicates, however, that this temperature factor may need modification.

Equation (1) can also be applied to recirculating filters if final effluent is recycled and mixed with incoming flow directly ahead of the filter. In this case the combined hydraulic load $Q_2 = Q_1\,(1+r)$ is used in the equation and the fraction of B.O.D. remaining, p_2, is related to the B.O.D. concentration of the mixed flow, L_m, so that:

$$p_2 = L_e/L_m = 10^{-KD/Q_1^{0.66}}, \tag{3}$$

p_2 can be converted to the normally used overall efficiency, p_3, which is related to incoming B.O.D. by the following equation:

VOL. 5 1961

$$p_2 = L_e/L_i = \frac{1}{\left(\frac{1+r}{p_1}\right)-r}. \tag{4}$$

Using a constant value of $K = 0{\cdot}3$ for single pass and recirculation conditions calculation showed that recirculation does not improve the degree of B.O.D. removal except at relatively high incoming hydraulic loads where a single pass filter would provide only 60 per cent efficiency or less. An increase in efficiency due to recirculation as predicted by the N.R.C. formula would therefore involve an increase in the value of K. So far no experimental data are available which would substantiate such an increase in K.

The fact that at any given hydraulic load and depth, a constant percentage of the applied B.O.D. is removed makes the trickling filtration process very similar to an adsorption process. Using a modified Freundlich adsorption isotherm this can be expressed as follows:

$$C_a/C_s = K'C_e^n, \tag{5}$$

where C_a = original concentration of adsorbate (applied B.O.D.) (mg/l.),
C_s = concentration of adsorbent (activated sludge or biological film) (mg/l.),
C_e = equilibrium concentration of adsorbate (effluent B.O.D.) (mg/l.),
K' = constant,
n = constant.

According to experimental data by Katz *et al.* (7) on activated sludge n was found to be equal to unity. If furthermore the concentration of adsorbent such as the active biological film in a trickling filter is assumed to be constant, equation (5) becomes

$$C_a = K'C_e, \tag{6}$$

which means that the B.O.D. remaining after a given contact time is a constant fraction of the applied B.O.D. The trickling filter process therefore appears as an adsorption process where hydraulic load and depth determine contact time and this in turn determines the level of efficiency at which the process operates.

REFERENCES

1. SCHULZE, K. L., Experimental vertical screen trickling filter, *Sewage Industr. Wastes* **29**, 4, 458 (1957).
2. SCHULZE, K. L., Load and efficiency of trickling filters. *J. W. P. C. F.* **32**, 3, 245 (1960).
3. HOWLAND, W. E., Flow over porous media as in a trickling filter. *Proc.* **12***th Ind. Wastes Conf., Purdue University, May* 1957, p. 435.
4. BLOODGOOD, D. E., TELETZKE, G. H. and POHLAND, F. G., Fundamental hydraulic principles of trickling filters. *Sewage Industr. Wastes* **31**, 3, 243 (1959).
5. WATER POLLUTION RESEARCH, 1952, Dept. of Scientific and Industrial Research, Her Majesty's Stationery Office, London (1953).
6. VELZ, C. J., A basic law for the performance of biological beds. *Sewage Wks. J.* **21**, 1, 31 (1949).
7. KATZ, W. J. and ROHLICH, G. A., A study of the equilibria and kinetics of adsorption by activated sludge. *Biological Treatment of Sewage and Industrial Wastes* (edited by MCCABE and ECKENFELDER) p. 66. Reinhold, New York (1956).

EFFECT OF SLUDGE LOADING AND DISSOLVED OXYGEN ON THE PERFORMANCE OF THE ACTIVATED SLUDGE PROCESS

H. E. Orford, H. Heukelekian and E. Isenberg

New Jersey Agricultural Station, Rutgers University

The purpose of this investigation is to obtain information concerning the relationships between loading and performance of the activated sludge process, using laboratory treatment units. The variables studied include: sewage B.O.D., mixed liquor volatile solids, aeration time, and mixed liquor dissolved oxygen. Temperature was eliminated from consideration by making all runs at 20°C. The influence of these variables on the process was measured by the effect on the sludge density indices, effluent B.O.D., effluent nitrates, sludge accumulation, and sludge oxygen demand.

VOL. 5 1961

EQUIPMENT

Six laboratory treatment units were designed as shown in Fig. 1. Six-inch lucite tubes with a capacity of 3 l. were used as aeration tanks, and were equipped with stainless steel mixing propellers driven through the bottom of the tank. The mixed liquor was siphoned from the aeration tank through a long $2^3/_4$ in. lucite tube which acted as a settling tank. The siphon discharged into a constant head chamber so

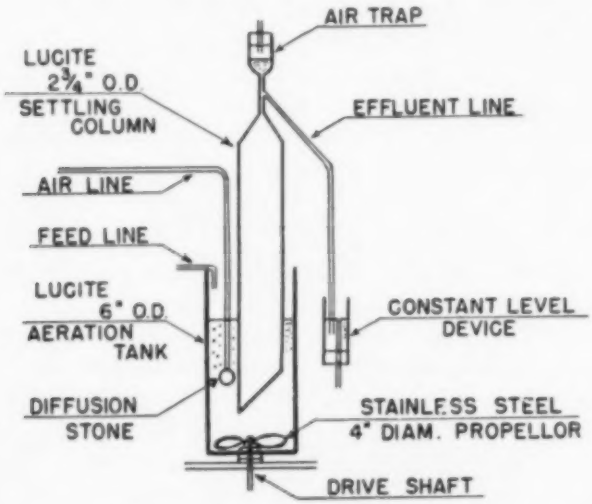


Fig. 1. Diagram of laboratory activated sludge unit.

that a constant volume was kept in the aeration tank at all times. Overflow from the constant head chamber then discharged into an effluent container.

Since the sewage was fed to the treatment units by individual peristaltic pumps, each connected to a time clock, a wide range of flow rates was available. Air supplied to each unit was measured by a calibrated orifice tube connected to a water manometer.

OPERATING PROCEDURES

Sewage for all experiments was collected from the primary sedimentation tank effluent of the N. J. State Hospital, Mariboro, a mental institution. This sewage is all domestic, is quite uniform in composition, and has a high concentration of B.O.D. and suspended solids. Weak sewage was easily obtained by dilution with tap water. A daily supply of sterilized sewage was stored in the intermittently stirred feed tank at 9.00 p.m. At 9.00 a.m. after 12 hr of continuous operation, the samples of effluent mixed liquor, and sewage feed were collected for the various tests. Since the sewage was not kept sterile in the storage tank, a sample was taken for B.O.D. analysis and the value obtained was used as the sewage B.O.D. for the entire 24-hr period.

The effluent sample could not be used directly for all analyses because of the heavy carry-over of mixed liquor solids during some periods of sludge bulking. The sampling procedure adopted was to determine suspended solids on the stirred effluent followed by one hour's settling before taking samples for the other analyses.

Sampling of the mixed liquor presented problems, because with bulky sludges the solids often almost filled the entire settling column. In order to make the mixed liquor sampling uniform for all tests, the aeration tank and settling tube volumes were mixed together before sampling. Thus, the detention times and sludge loading values for the mixed liquor were computed on the basis of 4-l. instead of the 3-l. volume of the aeration chamber proper.

After all samples were taken, appropriate amounts of mixed liquor were then wasted to maintain the solids content desired. In experiments where high sludge loadings were used, it was necessary to make mixed liquor suspended solids determinations and waste sludge twice a day.

METHODS AND PROCEDURES

The methods used in the analyses are those given in Standard Methods (6) with the following modifications and exceptions:

(1) The sludge density index test was performed in 500 ml cylinders and the solids content of the sludge tested was adjusted to between 1000 and 1500 mg/l. This latter was done to eliminate the effect of solids concentration on the final settling volume (4).

(2) The oxygen demand test of the activated sludge solids was performed by collecting samples of mixed liquor into a 30 ml centrifuge tube. The sludge concentration was adjusted to approximately 1000 mg/l. so that in subsequent washing

procedures all samples would be washed to an equal extent. Twenty-five milliliters of washed sludge was then placed in a Warburg apparatus, and oxygen uptake readings were taken over a period of 24 hr. The time between taking the sample and making the first oxygen uptake reading was never more than 1 hr.

(3) Dissolved oxygen. The Winkler dissolved oxygen method was modified so that smaller sample volumes of mixed liquor could be used. A sample for dissolved oxygen was pipetted directly from the aerator through a funnel containing glass wool into a 60 ml bottle containing 0·2 ml of alkaline iodide solution. Manganous sulfate, 0·2 ml, was then added and the sample shaken and acidified. Fifty milliliters of solution was then titrated with quarter strength sodium thiosulfate solution so that the dissolved oxygen was read in milliliters of thiosulfate used.

The sludge loading parameter used is pounds of B.O.D. applied per day per pound of volatile solids under aeration as recommended in *Units of Expression for Wastes and Waste Treatment* (7).

Statistical procedures

These studies were performed under a wide variety of activated sludge loading condition. Detention time in the aerator varied in the tests from less than 2 hr to over 33 hr, mixed liquor solids varied from less than 60 mg/l. to over 3500 mg/l., sewage strengths from less than 50 mg/l. to over 300 mg/l. and dissolved oxygen from over 6 mg/l. to zero. Analysis of the data obtained with the numerous combinations of sludge loading used was thus a complicated procedure.

VOL. 5 1961

All of the data reported was analyzed by an International Business Machine 650 electronic computer by a method of analysis devised by Cohen (2). It was decided that the logarithmic functions should be used to analyze the data; these functions fit a wide variety of curves, are simpler than parabolic or hyperbolic functions or others, and hold no apparent disadvantage over use of these other functions.

When the number of variables exceeds three, it is very difficult to show the data graphically. However, it is possible by multiple regression analysis to formulate a mathematical expression for the multidimensional plane representing the relationship between the variables. Such a formula puts all the evidence of a large number of observations into a single statement that expresses the extent to which differences in the dependent variable tend to be associated with differences in the independent variables.

Two coefficients were used to evaluate the various relationships obtained. First, the coefficient of correlation (two variables) or the index of multiple correlation (more than two variables) was used to evaluate what proportion of the variation in the dependent variable could be explained by the values of the independent variable(s). Second, the coefficient of regression was used to measure the slope of the regression line and thus indicate the relative effect of the independent variable upon the dependent variable.

In this report the correlation coefficient or correlation index, and partial correlation coefficient were found to be the most useful. When regression coefficients

are used they are given in the equation of the line of best fit and are the exponents of the various independent variables considered.

RESULTS

Comparison of raw, sterilized and synthetic sewage substrates

Before sterilized sewage was adopted as a substrate for this research, tests were made with raw, sterilized, and synthetic sewages to determine whether a suitable substitute could be obtained for raw sewage. Table 1 summarizes the results of activated sludge tests with each substrate. These tests were carried out for 6 days after allowing a 3-day sludge acclimitization period. The following synthetic sewage substrate (1) was used in the preliminary tests.

Proteose peptone,	0·3 g
Meat extract,	0·2 g
Urea,	0·05 g
Na_2HPO_4,	0·05 g
NaCl,	0·015 g
KCl,	0·007 g
$CaCl_2$,	0·007 g
$MgSO_4$,	0·005 g
Distilled water,	1·000 ml

Table 1. Summary of results of effect of type of sewage substrate

Measure examined	Synthetic sewage	Sterilized settled sewage	Settled sewage
Sludge loading (lb B.O.D. fed per day per lb mixed liquor volatile solids)	0·29	0·25	0·28
Sludge density index	1·88	1·48	1·75
Effluent B.O.D. (mg/l.)	6	7	7
Effluent nitrate nitrogen (mg/l.)	9·7	1·9	2·1
Oxygen uptake of activated sludge (mg O_2/hr/100 mg V.S.)	6·5	8·6	8·6

The results tended to show that either synthetic or sterilized sewage could be used in place of unsterilized settled sewage. Even though it required greater laboratory time and handling, sterilized sewage was selected because it was felt that all factors considered, it would react more closely like unsterilized sewage.

Effect of sludge loading on sludge density index

Previous experiments (3) showed that sludge production was best correlated to a sludge loading factor similar to "pounds of B.O.D. per day per pound of mixed liquor volatile solids under aeration". In these experiments, it was also found that

this simple sludge loading factor also could be used to evaluate the condition of the sludge as expressed by sludge density index.

Each test run used the six activated sludge units and lasted for about 2 weeks. After 4 days during which individual units became acclimitized to the particular loading imposed, daily analyses were made for 10 days. The sludge loading in each unit was kept as constant as possible but, despite all efforts, the individual loadings varied from day to day due mostly to flow variations in the peristaltic pumps and concentration changes in the mixed liquor solids. The greatest difficulty occurred at high loadings because of great accumulation in sludge solids and the fact that sludge was wasted once or twice per day.

Because of these daily variations in sludge loading, it was decided to analyze the data on the basis of each day's results and use an electronic computor to analyze the data.

Fig. 2 shows the logarithmic plot of sludge density index vs. the sludge loading in pounds of B.O.D. per day per pound of volatile sludge solids under aeration.

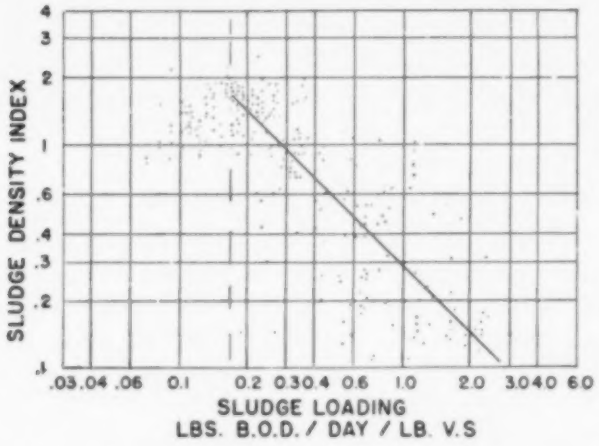


FIG. 2. Log relationship between sludge loading and sludge density index.

It shows that the maximum sludge density index occurs at a sludge loading of about 0·17. Above this loading the sludge density index decreases with increasing loading so that at loadings above 1·0 a sludge density index less than 0·3 can be expected.

It is interesting to note that at sludge loadings of $< 0{\cdot}17$, the sludge index decreases with increasing loadings. This effect has been previously observed (5) where decreasing sludge density indices were found below a loading of 0·25 pounds of B.O.D. per pound of sludge solids under aeration.

A regression analysis was made of the 207 analyses available between a sludge loading of 0·17 and 3·07 to obtain the relationship between sludge loading and sludge density index. The equation obtained was:

VOL. 5 1961

$$\text{S.D.I.} = \frac{0{\cdot}283}{\text{S.L.}^{0{\cdot}283}}, \tag{1}$$

where S.D.I. = sludge density index,
S.L. = sludge loading.

Although on a logarithmic plot the points seem scattered, a fairly high coefficient of correlation (−0·84) was obtained. This coefficient tells us that 70·5 per cent ($-0{\cdot}84^2 \times 100$) of the variation in the sludge index can be explained by the change in the sludge loading. The remaining 29·5 per cent of the variation is due to accumulated errors in measurement and analysis and, possibly, unconsidered factors which affect the sludge density index.

A multiple regression analysis was made on all the data shown in Fig. 2 to determine whether a better correlation could be obtained when each variable studied was considered. These included:

Sludge density index, S.D.I.
Sewage B.O.D. (mg/l.), B.O.D.
Mixed liquor volatile solids (mg/l.), V.S.
Aeration period, detention time (hr), D.T.
Dissolved oxygen (mg/l.), D.O.

The following multiple regression equation for these variables was obtained:

$$\text{S.D.I.} = 0{\cdot}0036 \frac{(\text{V.S.})^{0{\cdot}94}\ (\text{D.T.})^{0{\cdot}97}\ (\text{D.O.}+2)^{0{\cdot}13}}{(\text{B.O.D.})^{0{\cdot}67}} \tag{2}$$

The multiple correlation index for this equation (0·82) indicates that 67 per cent ($0{\cdot}82^2 \times 100$) of the variation in the sludge density index is explained by the four factors considered. Partial correlation coefficients of about 0·7 were obtained for the mixed liquor volatile solids, B.O.D., and detention time, but for dissolved oxygen the coefficient is only 0·13. This shows that B.O.D., mixed liquor volatile solids, and detention time have an important effect on the sludge density index and that the dissolved oxygen content is of little importance.

When the factors which make up the sludge loading, i.e., B.O.D., mixed liquor volatile solids, and detention time are considered separately (equation 2), they show slightly less correlation with the data than when the sludge loading factor is considered alone (equation 1).

Factors affecting effluent B.O.D.

The same loading factors used to study sludge density index were also used to evaluate the B.O.D. of the effluent. Data from 317 observations (Table 2) were analyzed as a whole, considering first the sludge loading factor and second the component variables separately, including mixed liquor dissolved oxygen (cols. 1 and 2). When the sludge loading factor was considered as the independent variable (col. 1) a correlation coefficient of 0·47 was obtained. While statistically significant, only

22 per cent of the variation of effluent B.O.D. was explained by the sludge loading factor. When the four variables were considered (col. 2) the correlation index was slightly higher, 0·50, explaining only 25 per cent of the variation in B.O.D. Of the four variables considered, only sewage B.O.D. and mixed liquor dissolved oxygen were statistically significant, each having a partial correlation coefficient of 0·33, explaining only 11 per cent of the variation in the effluent B.O.D.

The effluent B.O.D. increased with decreasing dissolved oxygen, increasing sewage B.O.D., and increasing sludge loading, but none of these effects were pronounced when considered over the entire range of loadings and mixed liquor dissolved oxygen.

TABLE 2. SUMMARY OF RESULTS ON FACTORS AFFECTING EFFLUENT B.O.D.

Column No.	1	2	3	4
Number of observations	317	317	53	264
Mean values				
Eff. B.O.D. (mg/l.)	8·3	8·3	14·4	7·0
Sewage B.O.D. (mg/l.)	125	125	94	131
Detention time (hr)	5·5	5·5	5·3	5·6
D.O. (mg/l.)	3·2	3·2	0·3	3·8
Sludge loading ratio	0·38	0·38	—	—
Mixed liquor V.S. (mg/l.)	1010	1010	—	—
Correlation coefficient	0·47	0·50	0·54	0·56
Explained variation (%)	22	25	29	31
Range				
Eff. B.O.D. (mg/l.).	1–71	1–71	—	—
Sewage B.O.D. (mg/l.)	15–368	15–368	—	—
Detention time (hr)	1·3–33·1	1·3–33·1	—	—
D.O. (mg/l.)	0·0–7·6	0·0–7·6	0·0–0·5	0·6–7·6
Sludge loading ratio	0·07–2·35	0·07–2·35	—	—
Mixed liquor V.S.	330–3115	330–3115	—	—

Col. 1 – simple correlation between effluent B.O.D. as the dependent variable and sludge loading as the independent variable.

Cols. 2, 3, 4—multiple correlation–effluent B.O.D. as the dependent variable and sewage B.O.D., detention time, dissolved oxygen and mixed liquor volatile solids as independent variables.

Since dissolved oxygen showed some effect on effluent B.O.D., the data were divided into two groups, those observations where the dissolved oxygen was 0·5 mg/l. or less and those where the dissolved oxygen was 0·6 mg/l. or greater (cols. 3 and 4). The results now show a somewhat greater degree of correlation and about 30 per cent of the variation in effluent B.O.D. is explained by the variables considered. The most significant observation is that the average effluent B.O.D. is 14·4 mg/l. in the low dissolved oxygen range and only 7·0 mg/l. in the high dissolved oxygen range.

VOL. 5 1961

Thus, when the mixed liquor dissolved oxygen is close to zero there is a pronounced inverse relationship between effluent B.O.D. and dissolved oxygen content. This was also shown by the fact that at the low dissolved oxygen levels, the regression coefficient for dissolved oxygen was −44 while at the higher dissolved oxygen levels it was −0·75.

Factors affecting effluent nitrate

The combined nitrate and nitrite content of the effluent was measured to determine the important factors affecting nitrate production. When the nitrate content was plotted against the sludge loading ratio, a poor correlation was obtained. This was to be expected because other factors such as ammonia and dissolved oxygen content of the sewage would have an important effect upon the nitrate level of the effluent.

A multiple regression analysis of the data was made and the statistical results are given in Table 3.

TABLE 3. SUMMARY OF STATISTICAL CONSTANTS FOR FACTORS AFFECTING EFFLUENT NITRATE CONTENT

Number of observations	177
Index of correlation (overall)	0·73
Partial correlation coefficients	
B.O.D. (sewage B.O.D.), mg/l.	−0·39
D.T. (aeration detention time) hr	+0·24
NH_3 (sewage ammonia nitrogen), mg/l.	+0·47
D.O. (mixed liquor dissolved oxygen—D.O. +2), mg/l.	+0·67
V.S. (mixed liquor volatile solids), mg/l.	+0·28
NO_3 (effluent nitrate nitrogen) mg/l.	—

The following multiple regression equation shows the relationships between the various factors:

$$NO_3 = \frac{7{\cdot}48 \times 10^{-6}(\text{D.T.})^{1{\cdot}01} \times (NH_3)^{1{\cdot}02} \times (\text{D.O.}+2)^{2{\cdot}56} \times (\text{V.S.})^{1{\cdot}26}}{\text{B.O.D.}^{1{\cdot}33}} \qquad (3)$$

The factors analyzed showed an overall correlation index of 0·73, thus explaining 54 per cent ($0{\cdot}73^2 \times 100$) of the variation found in the nitrate content of the effluent. From the partial correlation coefficients obtained it was determined that all the factors studied were statistically significant. The dissolved oxygen level had the greatest effect on nitrification followed by sewage ammonia content, B.O.D., mixed liquor volatile solids, and aeration time. Since dissolved oxygen and ammonia had such an important effect on the nitrate production it is not surprising that a poor correlation was obtained between nitrates and sludge loading.

Factors affecting production of activated sludge solids

Activated sludge solids production has been expressed (3) as pounds of volatile solids produced per pound of B.O.D. fed. For this work the expression was changed

to pounds of volatile solids accumulated per pound of B.O.D. removed. It was believed that this expression would be more valid, particularly since the high sludge loadings sometimes used produced effluents with high B.O.D. values. This effluent B.O.D. would then be lost for the production of sludge solids.

It was not possible to obtain definitive results on the sludge accumulation on the basis of daily sampling. Consequently, the solids accumulation was measured for the duration of a run and then compared with the averages of the sludge loading and other factors.

The relationship between solids accumulation and sludge loading is shown in Fig. 3, in which the volatile solids accumulation per pound of B.O.D. removed

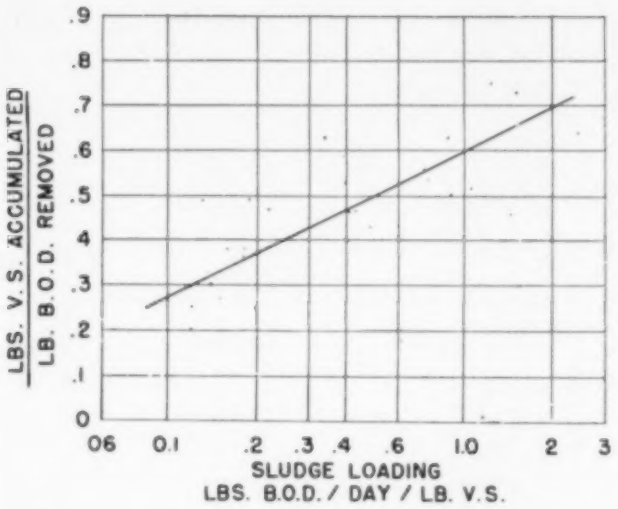


FIG. 3. Effect of sludge loading on sludge solids accumulation.

is plotted against the log of the sludge loading. A regression analysis of the data was made and the following equation of the line of best fit was obtained.

$$S.A. = 0{\cdot}313 \log S.L. + 0{\cdot}593,$$

where S.A. = sludge accumulation in pounds of volatile solids per pound of B.O.D. removed,

S.L. = sludge loading in pounds of B.O.D. per day per pound of volatile solids under aeration.

The coefficient of correlation was 0·79 indicating that sludge loading accounted for 62 per cent (0·792 × 100) of the variation in sludge accumulation, and was found to be highly significant statistically.

The sludge accumulation found was higher than that obtained by others (3), over a comparable range of loadings. Part of the difference can be explained by the fact that in the former case, the amount of B.O.D. fed was used in evaluating sludge accumulation while in this study the amount of B.O.D. removed is used.

VOL. 5 1961

In the sludge loading ranges normally encountered, the sludge accumulation found is about 20 per cent greater than that established before (3), while in the very low sludge loading ranges, the difference is greater.

Factors affecting activated sludge oxygen demand

Oxygen demand rates of the activated sludges were measured with the Warburg respirometer. The sludges were double-washed by alternate centrifuging and washing with dechlorinated tap water to wash out any oxygen demanding materials in the substrate. Thus, the oxygen uptake rates obtained reflect the oxygen demand of the sludge itself.

A simple correlation analysis was made for the 169 sets of observations for which sludge oxygen demand rates were measured. Data for the analysis is plotted in Fig. 4, which also shows the line of best fit obtained from the analysis. This line has the follcwing equation:

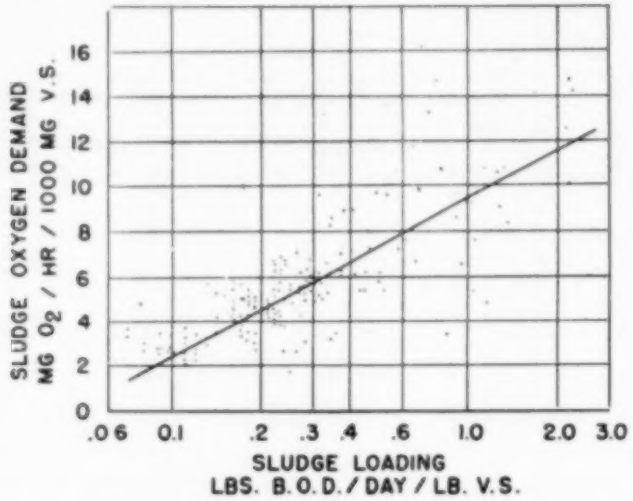


FIG. 4. Influence of sludge loading on sludge oxygen demand.

$$S.O.D. = 6{\cdot}52 \log S.L. + 9{\cdot}361,$$

where S.O.D. = sludge oxygen demand in mg of oxygen demand per hour per 1000 mg of sludge volatile solids,

S.L. = sludge loading pounds of B.O.D. per day per pound of sludge volatile solids.

The data shows an increasing sludge oxygen demand as the sludge loading increases. However, since a semi-logarithmic relationship is involved, there is a decreasing rate of increase in sludge oxygen demand.

A simple correlation analysis for the data gave a high correlation coefficient of 0·82, indicating that 67 per cent of the variation in sludge oxygen demand is explained by the sludge loading factor.

In order to check whether the dissolved oxygen level, during the aeration of the activated sludge, was a factor in the sludge oxygen demand, a multiple correlation analysis was made (Table 4). The overall correlation index to the sludge oxygen demand for the four variables considered (B.O.D., aeration time, mixed liquor volatile solids, and mixed liquor D.O.) is 0·82, which is the same as the correlation index found for the sludge loading alone. The three factors B.O.D., aeration detention time, and mixed liquor volatile solids, which combine to give the sludge loading factor, all have high partial correlation coefficients. These are all highly significant statistically while the low partial correlation coefficient for dissolved oxygen has no statistical significance. Thus, from the standpoint of sludge oxygen demand the use of the simpler relationship with sludge loading is to be preferred.

TABLE 4. SUMMARY OF STATISTICAL CONSTANTS FOR FACTORS AFFECTING THE SLUDGE OXYGEN DEMAND OF WASHED ACTIVATED SLUDGE

Number of observations 169
Correlation index (overall) 0·82

Variable	Units	Range	Geom. mean	Partial cor. coef.
Sewage, B.O.D.	mg/l.	15–269	90	+0·79
Aeration detention time	hr	1·4–30·9	6·9	−0·70
Mixed liquor				
Vol. solids	mg/l.	380–3340	1100	−0·64
Mixed liquor D.O.	mg/l.	0–7·6	2·6	−0·09

VOL. 5 1961

SUMMARY

Some factors affecting the performance of the activated sludge process were studied on a laboratory scale using six individual treatment units.

The variables studied over a wide range included sewage B.O.D., mixed liquor volatile solids, aeration time, and mixed liquor dissolved oxygen. The influence of these variables on the process was measured by the effect on sludge density index, effluent B.O.D., effluent nitrates, sludge accumulation, and sludge oxygen demand.

It was found that the sludge loading expressed as lb./B.O.D./day/lb. of mixed liquor volatile solids was the best parameter for evaluating the sludge density index, which decreased with increasing sludge loading. When dissolved oxygen was considered with the variables which make up sludge loading, there was no improvement in the correlation with sludge density index. A regression analysis was made on data to develop equations which show the relative importance of the variables on the sludge density index.

The effluent B.O.D. increased with sludge loading, but a slightly better correlation was obtained when mixed liquor dissolved oxygen was included in the variables considered. Dissolved oxygen became a very important factor at low levels below

0·5 mg/l. In this range the average effluent B.O.D. value was twice as great as that found when the dissolved oxygen ranged from 0·6 to over 6 mg/l.

The effluent nitrate content was found to be affected by the mixed liquor dissolved oxygen and the ammonia content of the sewage. Consequently, the correlation obtained with sludge loading was not high. A multiple regression analysis showed that the sewage ammonia content and the factors contributing to sludge loading were of about equal importance in their effect on the effluent nitrate levels.

Sludge accumulation was measured as pounds of volatile solids produced per pound of B.O.D. removed. The sludge solids produced increased with the sludge loading, and good correlation was obtained for the relationship developed.

The oxygen demand rate of the washed activated sludge was found to increase with the sludge loading, and a good correlation between these two variables was obtained. When the factors making up the sludge loading expression were considered separately along with the mixed liquor dissolved oxygen no better correlation was obtained. Thus, the dissolved oxygen level had little influence on the oxygen demand of the activated sludge solids.

CONCLUSIONS

From the results obtained from a laboratory scale study of the activated sludge treatment of sewage, it can be concluded that:

(1) Heat sterilized sewage can be used for laboratory studies with activated sludge without seriously affecting the results obtained.

(2) The sludge density index decreases with increasing sludge loading above a loading of 0·17 lb./B.O.D./day/lb. of volatile solids under aeration. Below this loading the sludge index decreases with decreasing sludge loading.

(3) The sludge density index is only slightly increased with increasing levels of dissolved oxygen in the mixed liquor.

(4) The effluent B.O.D. showed a low level of correlation with the factors making up the sludge loading and with mixed liquor dissolved oxygen above 0·6 mg/l. At dissolved oxygen levels below 0·5 mg/l., the effluent B.O.D. sharply increases with decreasing dissolved oxygen.

(5) The nitrate content of the activated sludge effluent increases significantly with increasing sewage ammonia content and mixed liquor dissolved oxygen, and decreases as the sludge loading increases.

(6) The production of activated sludge solids increases with increasing sludge loading. No important relationship was observed between mixed liquor dissolved oxygen and sludge solids production.

(7) The oxygen demand rate of washed activated sludge solids increases with increasing sludge loading. No important relationship was observed between mixed liquor dissolved oxygen and sludge oxygen demand.

REFERENCES

1. BUTTERFIELD, C. T., RUCHHOFT, C. C. and MCNAMEE, P. D., Studies of sewage purification—VI. Biochemical oxidation of sludges developed in pure cultures of bacteria isolated from activated sludge. *Sewage Wks. J.* **9**, 2, 173 (1937).
2. COHEN, A., *Multiple Regression Analysis*. 650, Program Library, File Number 6.0.001, International Business Machines, New York.
3. HEUKELEKIAN, H., ORFORD, H. E. and MANGANELLI, R. M., Factors affecting the quantity of sludge production in the activated sludge process. *Sewage Industr. Wastes* **23**, 8, 945 (1951).
4. ISENBERG, E. and HEUKELEKIAN, H., Sludge volume index. *Wat. Sewage Wks* **106**, 12, 525 (1959).
5. LOGAN, R. P. and BUDD, W. E., Effect of B.O.D loadings on activated sludge plant operation. *Biological Treatment of Sewage and Industrial Wastes* vol. 1. *Aerobic Oxidation* (edited by MCCABE and ECKENFELDER). Reinhold, New York (1956).
6. *Standard Methods of Water and Sewage Analysis*, 10th ed.
7. *Units of Expression for Wastes and Water Treatment*. Manual of Practice No. 6, Federation of Sewage and Industrial Wastes Assn.

VOL.
5
1961

MATHEMATICAL FORMULATION OF THE BIOLOGICAL OXIDATION PROCESS

BROTHER JOSEPH MCCABE

Manhattan College

VOL. 5 1961

THIS paper suggests a mathematical model to describe the removal of B.O.D. from waste waters during a biological oxidation process. Such B.O.D. removal can be attributed to: (a) biosorption—by initial removal during a brief period of contact with biologically active sludge; (b) sludge growth—by microbiological assimilation for respiration and the synthesis of new cells. The beginning of this paper formulates

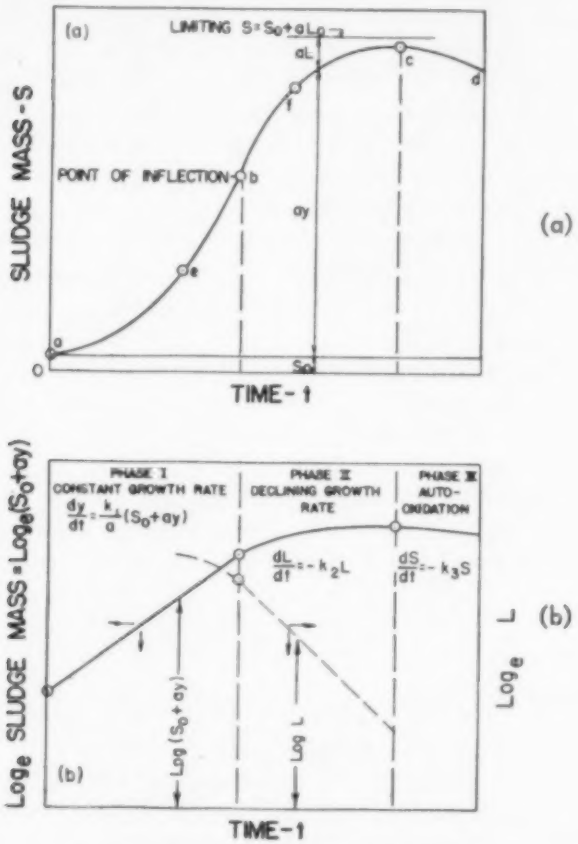


FIG. 1. Ideal activated sludge growth curve.

the removal of B.O.D. due to sludge growth alone, without initial removals; initial B.O.D. removals are considered later.

Garrett (1, 2) showed that the rate of growth for activated sludge can be represented as a discontinuous function, constant at high B.O.D. values and directly proportional to the remaining B.O.D. present at low concentrations. These two phases constitute the sigmoidal portion of the growth curve exhibited by biological populations limited in space or food. Fig. 1(a) is an ideal growth curve for a batch oxidation started at a high loading ratio but assuming no lag phase. This growth curve exhibits three distinct phases:

phase I, a constant growth phase (*a*–*b*);
phase II, a declining growth phase (*b*–*c*);
phase III, an auto-oxidation phase (*c*–*d*).

Phase III will not be considered in this paper, since no significant B.O.D. reductions are associated with it. The linear portion of the sigmoidal curve (e–f) approximates an arithmetic growth rate within the limits of linearity from *e* to *f*.

Phases I and II constitute an S-curve whose mathematical formulation as a continuous function has been presented by Fair and Moore (3) together with its approximation as a two-phase discontinuous function. The evaluation of the two rate constants for the single continuous function is rather tedious. On the other hand the solution of the two discontinuous functions is simple and direct and provides sufficient accuracy for the formulation of engineering data.

At the beginning of the oxidation period let:

S_0 = the initial sludge concentration (biological cellular material),
L_0 = the total amount of organic matter initially present that can be removed as a limit of the oxidation process, measured as B.O.D.

At any time (t) let:

S = the sludge concentration present,
ΔS = $S-S_0$ = the increase in sludge concentration,
L = the oxidizable B.O.D. remaining,
y = the B.O.D. removed,
y_t = the B.O.D. removed by biosorption,
$y-y_t$ = the B.O.D. removed by assimilation for respiration and synthesis.

Let a be the fraction of the B.O.D. removed which is synthesized to sludge at any time. Then

$$\text{if } y_t = 0, \quad ay = \Delta S \text{ and } S = S_0 + \Delta S = S_0 + ay \tag{1}$$

PHASE I: LOGARITHMIC GROWTH PHASE (*a* to *b*)

The two-phase formulation assumes that the growth rate in Phase I, in the presence of a high B.O.D. concentration, is limited only by the minimum generation time of the organisms themselves for the temperature used. Thus regular and maximum multiplication occurs, and the growth rate is a constant maximum. Phase I is therefore defined by a constant rate of sludge increase:

$$dS/dt = d(S_0+ay)/dt = K_1S \tag{2}$$

which integrates to

$$\log_e(S_0+\Delta S)/S_0 = K_1t \tag{3}$$

or, in terms of B.O.D. removal (y), and assuming for the present no initial removal,

$$\log_e(S_0+ay)/S_0 = K_1t \tag{4}$$

where K_1 is the logarithmic growth rate (time)$^{-1}$ for natural logarithms; the constant for common logs is k_1.

The plot of log (S_0+ay) against t (shown as solid line in Fig. 1(*b*)) is a straight line function within Phase I (*a*-*b*) and the slope defines k_1. The decreasing slope of the line after point *b* indicates the declining growth rate of Phase II.*

PHASE II: DECLINING GROWTH PHASE (*b* to *c*)

The two-phase formulation assumes that the influence of decreasing B.O.D. concentration begins to limit growth at some transition point (*b*), after which the growth rate, and consequently the B.O.D. removal rate, are proportional to the B.O.D. remaining. Phase II is therefore defined by the rate of B.O.D. decrease.

$$-dL/dt = K_2L \tag{5}$$

which integrates to

$$\log_e L/L_0 = -K_2t \tag{6}$$

K_2 is the logarithmic B.O.D. removal rate (time)$^{-1}$ for natural logarithms. The constant for common logs is k_2. The plot of log L against t (shown by the dashed line in Fig. 1(*b*)) is a straight line function within Phase II and the slope defines k_2. It is often convenient to plot log L/L_0 against t to compare oxidations under various conditions.

While equation (5) often describes the course of B.O.D. removal during the oxidation of simple substrates within Phase II, the heterogeneous mixtures encountered in sewage or industrial wastes usually yield a progression of decreasing rate constants as the less readily assimilable substances are removed; first-order kinetics do not therefore apply. The evaluation of progressive K_2 values for such complex wastes is discussed by Eckenfelder (4) below.

ARITHMETIC GROWTH PHASE

The linear portion of the ideal growth curve (*e*-*f*) approximates an arithmetic growth phase and within the limits of linearity the associated B.O.D. removals

* As noted by Garrett and Sawyer (2), within any portion of Phase I where the per cent increase in sludge mass is not greater than 100 per cent ($\Delta S < S_0$), relatively little error is introduced by letting $S = S_a$, the average sludge concentration over the range under consideration. Then if $t_0 = 0$ equation (2) becomes

$$\Delta y = K_1S_at/a \tag{7}$$

which can also be used as an approximate expression for the linear portion (*e*-*f*) of the growth curve if $\Delta S < S_0$.

VOL. 5 1961

will be zero order as given by equation (7). When batch oxidations are started at high concentrations of micro-organisms (low loading ratios), the initial portion of the sigmoidal curve (*a*–*e*) is frequently masked.

The lower substrate concentration limiting linearity is defined by Wuhrmann (see discussion, p. 287) as that when diffusion rates become less than fixation rates. Using quantitative determination of the substrates themselve, linear removal was determined down to very low concentrations. Non-linear removals were reported when B.O.D. or organic nitrogen were used as parameters (5).

Point of discontinuity (*b*)

At the transition point (*b*) the B.O.D. removal rates due to Phase I and Phase II are just equal. This point of discontinuity between Phase I and Phase II can be estimated approximately from the inflection point in Fig. 1(a) and from the linearity in Fig. 1(b). More exact calculations can be obtained by equating equations (2) and (5) multiplied by a

$$L_b/S_b = L_b/(S_0+ay_b) = K_1/aK_2 \tag{8}$$

or
$$K_1(S_0+ay_b) = aK_2(L_0-y_b)$$

and

$$y_b/L_0 = \frac{K_2-K_1S_0/aL_0}{K_1+K_2} \tag{9}$$

$$L_b/L_0 = \frac{K_1+K_1S_0/aL_0}{K_1+K_2} \tag{10}$$

Then, from equation (4)

$$t_b = (1/K_1)\log_e\frac{S_0+ay_b}{S_0} = 1/K_1\log_e S_b/S_0 \tag{11}$$

By equation (8) K_2 varies inversely with the L_b/S_b ratio for any given oxidation system since K_1 and a are taken as constants of the system.

Sludge growth ratio

The assumption that the fraction of B.O.D. removed which is synthesized to sludge (sludge growth ratio a) is constant at any time is inexact, but an approximate value of 2/3 may be sufficiently accurate for practical applications (6). The ratio actually varies with the age of the sludge, the quantity of available nitrogen, etc., and the determination is complicated by biosorption and auto-oxidation.

From data on B.O.D. removal and associated sludge build-up, the a value can be approximated as the slope of a plot of $\Delta S/S_a$ vs. y/S_a. The intercept of the slope at zero removal is interpreted as an approximation of the sludge fraction consumed during continual endogenous respiration, but this assumption must be used with caution to estimate the amount of stored material initially removed. Determination of a from oxygen utilization data is preferable because it avoids the interference of inert solids in the measurement of sludge growth.

Active sludge fraction

The above equations have been presented with the definition of S as the mass of active biological cellular matter and it is necessary to estimate the initial mass of active bacteria to predict B.O.D. removals accurately. On the other hand, an estimation of gross sludge solids is required to predict the amount of excess sludge for ultimate disposal.

A crude estimate of the active biological sludge solids can be obtained from the volatile solids content, but a gradual build- up of inert organic material frequently interferes with this estimate. McKinney (6) reports that organic nitrogen in the biological solids can be used to indicate active solids unless inert nitrogen-containing compounds accumulate. The oxygen uptake rate will provide information on the active sludge fraction when the constants discussed above are determined.

EXAMPLES OF THE TWO-PHASE FORMULATION

The two-phase formulation of B.O.D. removal during bio-oxidation is defined by the constants a, S_0, L_0, K_1 and K_2. Examples of this method of analysis given

TABLE 1. TWO-PHASE ANALYSIS OF SIMPLE SUBSTRATE BY ACTIVATED SLUDGE*

	Peptone substrate (Fig. 2)			Glucose substrate (Fig. 3)	
	A	*B*	*C*	*B*	*C*
Feed B.O.D., mg/l.	340	640	136	560	1120
Initial B.O.D. = L_0, mg/l.	225	450	900	370	740
Initial T.S.S., mg/l.	1340	670	1340	550	1100
V.S., as % T.S.S.	72·4	72·4	72·4	76·3	76·3
a. mg V.S.S./l./mg B.O.D./l.	0·6	0·6	0·6	0·7	0·7
Initial V.S.S. = S_0, mg/l.	970	485	970	420	840
L_0/S_0	0·23	0·93	0·93	0·88	0·88
Initial removal = y_i, mg/l.	0	0	0	77	141
k_1 (common logs), hr^{-1}	0·05	0·05	0·05	0·053	0·053
k_2 (common logs), hr^{-1}	0·488	0·19	0·19	0·83	0·83
K_1 (natural logs), hr^{-1}	0·115	0·115	0·115	0·122	0·122
K_2 (natural logs), hr^{-1}	1·12	0·435	0·435	1·9	1·9
t_b, hr	0·3	2·0	2·0	2·5	2·5
At point b:					
y_b, mg/l.	55	187	374	312	624
L_b, mg/l.	170	263	526	58	116
S_b, mg/l.	1003	600	1200	638	1276
L_b/S_b	0·17	0·44	0·44	0·91	0·91
At point b':					
M_b', mg/l.				235	483
L_b', mg/l.				135	257
S_b', mg/l.				585	1178
L_b'/S_b'				0·23	0·22

* Experimental data after Garrett (1).

VOL. 5 1961

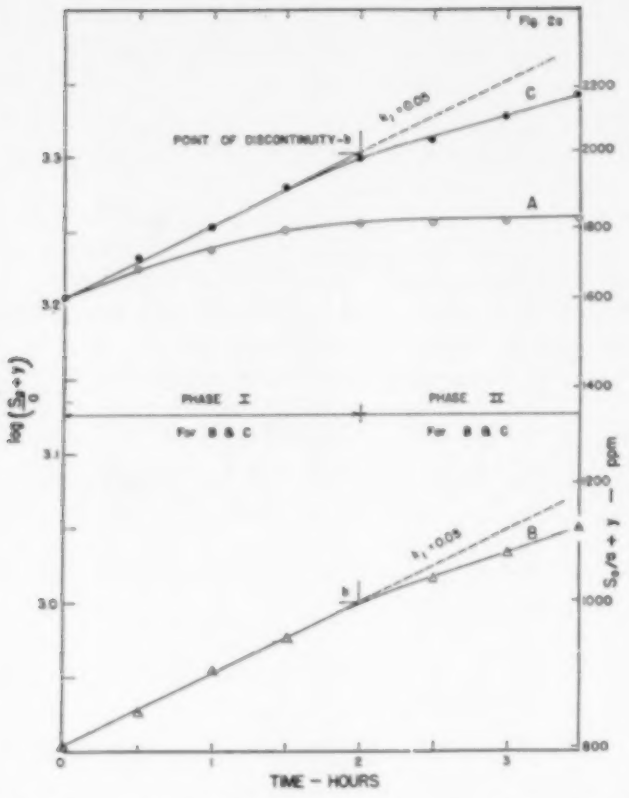


FIG. 2a. B.O.D. removal by activated sludge-peptone substrate. Experimental data after Garrett (1).

in the following figures show the relative simplicity of this method of analyzing data. $(S_0/a+y)$ and L values were plotted against time on semi-log paper and the point of discontinuity was approximated by eye. The slopes of best fit for the experimental values were used for initial estimates of k_1 and k_2; the co-ordinates of the point of discontinuity were calculated using equations (8) to (11) and the two rate constants were revised.

Simple substrates

Examples for two simple substrates are shown in Figs. 2 and 3 using the experimental data obtained from Garrett's (1) curves of B.O.D. remaining and oxygen utilization by activated sludge at 25°C. From his oxygen utilization curves approximate a values of 0·6 for glucose and 0·7 for peptone were estimated on the basis of volatile solids synthesized per unit of 5-day B.O.D. assimilated. Pertinent information is listed in Table 1.

An example without apparent initial removal is given in Fig. 2. The feed B.O.D. was reduced by a third after the peptone substrate was mixed with the sludge. Since

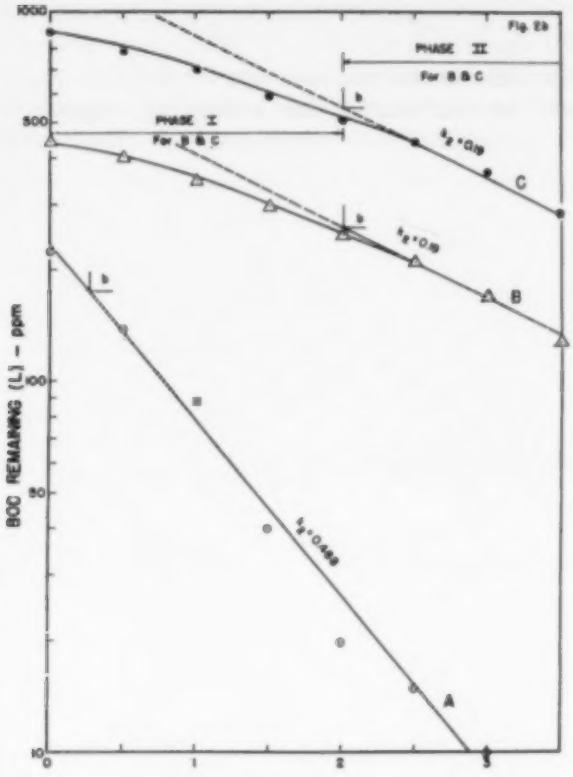


FIG. 2b.

no further adsorption was apparent thereafter, assimilation was assumed to be responsible for all further removal. Curves B and C with identical L_0/S_0 ratios, four times that of curve A, followed parallel K_1 and K_2 rates. They therefore exhibited a more extended Phase I than did Curve A. The K_1 rate was constant for the system and consequently, according to equation (8), K_2 was inversely proportional to the L_b/S_b ratio at the transition point.

An example of the course of B.O.D. removal attributed to both biosorption and assimilation is given in Fig. 3. In addition to the immediate B.O.D. reductions (one-third of feed B.O.D.) after the glucose substrate was mixed with sludge, further biosorption occurred for about 1 hr. After this point B.O.D. removals in Phase I were attributed to assimilation.

With equal L_0/S_0 ratios it would be expected that both curves would have equal K_1 and K_2 values as shown in Fig. 2. Sufficient points on each total removal curve were available to estimate the K_1 value. The parallel line drawn through S_0/a was taken to indicate the course of removal due to assimilation. The intercept between the parallel lines of total removal and removal due to assimilation was used to

VOL. 5 1961

estimate the amount of B.O.D. adsorbed by the sludge after feeding (y_i). By this estimation curve C, with twice the S_0 (and twice the L_0 also) shows twice the y_i; in each case the total amount was removed in about 1 hr.

It would be expected that additional organic material would be adsorbed as sludge is synthesized, and perhaps this explains the slightly increasing calculated

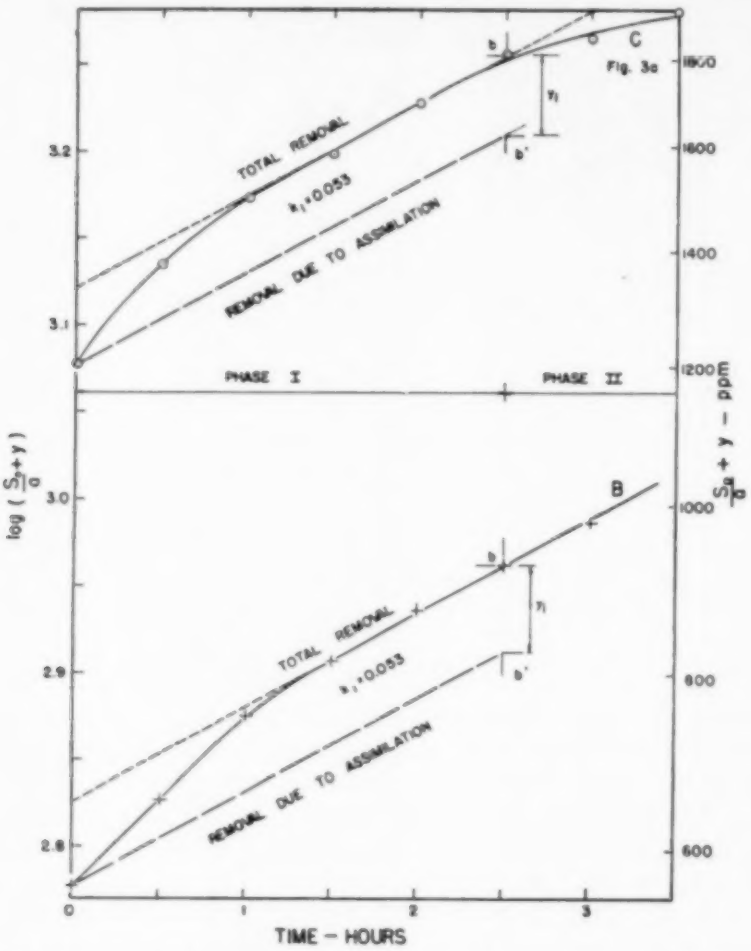


FIG. 3a. B.O.D. removal by activated sludge–glucose substrate. Experimental data after Garrett(1).

K_1 values often found for each interval when high sludge growth is obtained, even after appreciable early bisorption by the sludge inoculum has seemingly reached equilibrium. This would make the use of an average K_1 value an approximation, but one which is within the limits of error for most engineering data.

Engelbrecht (7) showed that the adsorbed material contributed to the mass of biological solids and believed that its utilization by the micro-organisms permitted

adsorption to continue until the substrate concentration became limiting; then the final amount of adsorbed material was metabolized causing a rapid decrease in the mass of the solids. His data showed that the ratio of biological solids to B.O.D. removed increased sharply until a region considered here to be near the point of discontinuity (*b*), and dropped off thereafter. In this formulation it is therefore

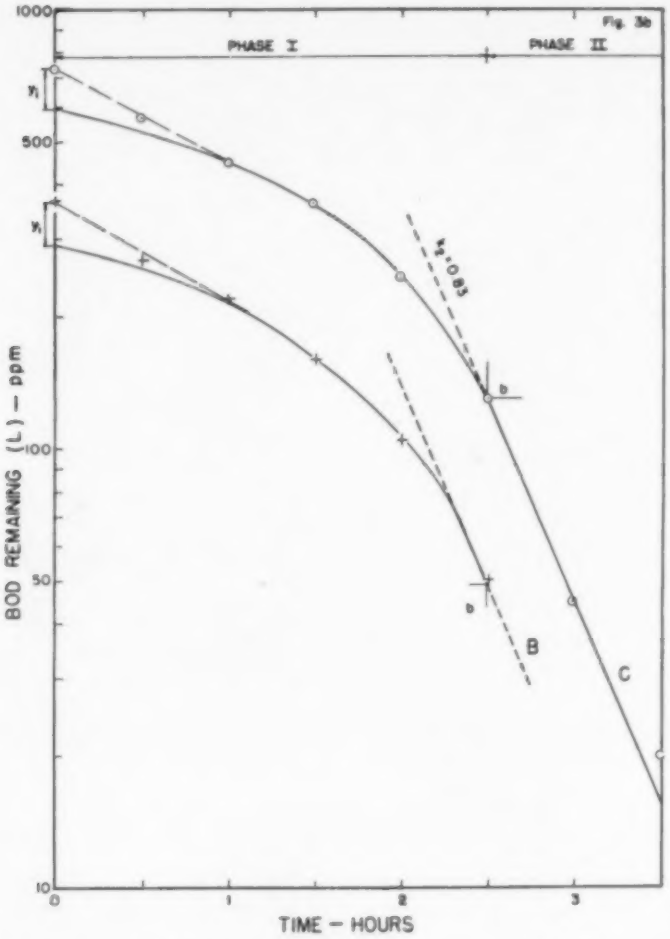


FIG. 3b. B.O.D. removal by activated sludge–glucose substrate. Experimental data after Garrett (1).

assumed that the amount of adsorbed material initially removed is metabolized in Phase II immediately after the point of discontinuity. Corrected B.O.D. values were therefore used for the formulation of removals caused by assimilation during Phase I:

$$L_0' = L_0 - y_i \tag{12}$$

$$y' = y - y_i \tag{13}$$

VOL. 5 1961

With these corrections the transition points were calculated as shown in Table 1 and Fig. 3. Examination of oxygen utilization curves also showed that the points of discontinuity occurred at about 2½ hr and that curve *B*, for which insufficient B.O.D. values were available in Phase II, did exhibit a K_2 value equal to the Phase II rate for curve *C*.

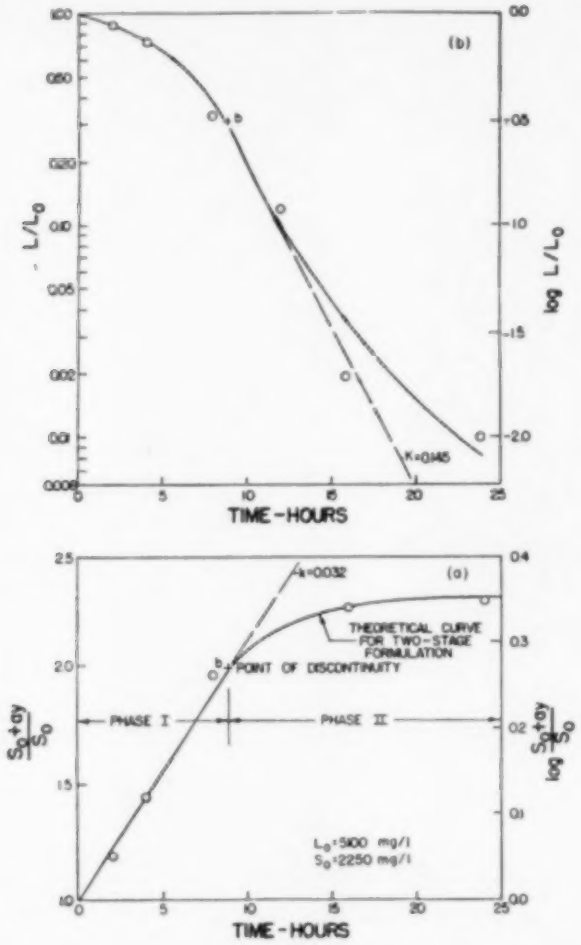


FIG. 4. B.O.D. removal by activated sludge-pharmaceutical waste.

Further analysis will be required to ascertain when adsorbed material is assimilated. The formulation of Phase II B.O.D. reductions as a function of the actual *L* values, and not of the B.O.D. remaining in solution *plus* that adsorbed, fits the data well.

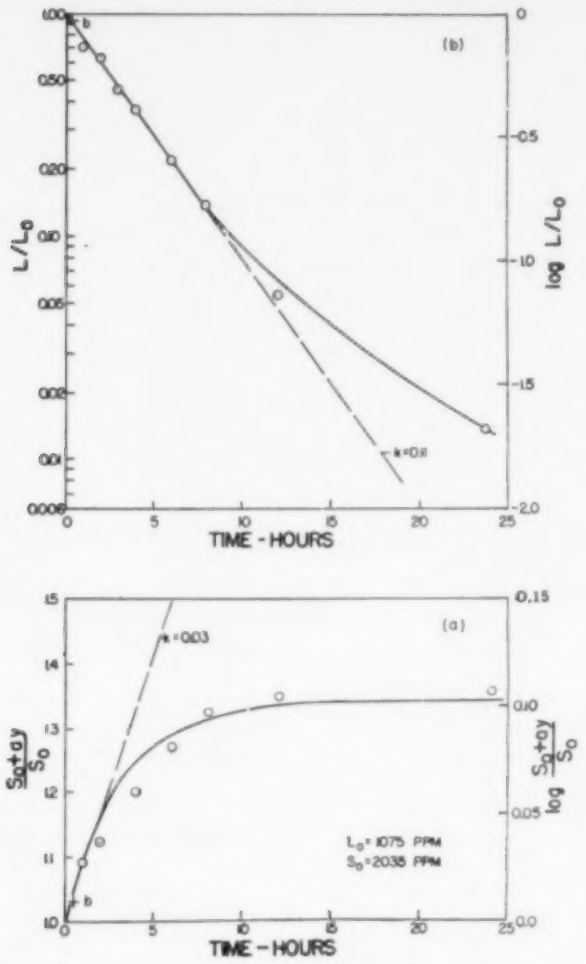


FIG. 5. B.O.D. removal by activated sludge-pharmaceutical waste

Industrial waste

Examples of the two-phase formulation of B.O.D. removal from an industrial waste with a decreasing rate constant in Phase II are presented in Figs. 4 and 5 and Table 2. These represent two batch oxidations (20°C±1°C) of wastes from a pharmaceutical plant on different dates. The S_0 values were gross suspended solids and the experimental values were field B.O.D. data. The retardant formula of Fair *et al.* (8) was used to obtain the decreasing rate in Phase II, replacing K_2 by $K_2/(1+mt)$ where $t_b = 0$ and m is a coefficient of retardation. Other formulations for the solution of the non-uniformity coefficient (4) would probably be preferred, but the few points available for Phase II in Figs. 4 and 5 do not justify further refinement.

VOL. 5 1961

TABLE 2. TWO-PHASE ANALYSIS OF BIOLOGICAL OXIDATION OF A PHARMACEUTICAL WASTE

	Fig. 4	Fig. 5
a	0·55	0·55
S_0	2250 ppm	2035 ppm
L_0	5100 ppm	1075 ppm
K_1	0·069 $(hr)^{-1}$	0·069 $(hr)^{-1}$
K_2	0·333 $(hr)^{-1}$	0·253 $(hr)^{-1}$
m	0·05	0·05
tb	9·0 hr	0·4 hr
Lb	1580 ppm	980 ppm

The two-phase formulation fits laboratory and field data fairly well and the examples show the relative simplicity of this method of formulating B.O.D. removals during batch biological oxidations.

REFERENCES

1. GARRETT T. M., Kinetics of removal of soluble B.O.D. by activated sludge. Unpublished Doctoral thesis, M. I. T. (1951).
2. GARRETT T. M. and SAWYER C. N., Kinetics of removal of soluble B.O.D. by activated sludge. *Proc. 7th Ind. Waste Conf. Purdue University*, Ext. Service 79, 51 (1952).
3. FAIR G. M. and MOORE E. W., Mathematical formulation of the course of digestion. *Sewage Wks. J.* **4**, 429 (1932).
4. ECKENFELDER W. W., this volume, p. 277.
5. WUHRMANN K., Factors affecting efficiency and solids production in the activated sludge process. *Biological Treatment of Sewage and Industrial Wastes*, vol. 1, p. 49. Reinhold, New York (1956).
6. MCKINNEY R., this volume, p. 1.
7. ENGELBRECHT R. S., Energy relations in the activated sludge process. Unpublished Doctoral thesis, M. I. T. (1954).
8. FAIR G. M., MOORE E. W. and THOMAS H. A., JR., The natural purification of river muds and pollutional sediments. *Sewage Wks. J.* **13**, 270 (1941).

APPLICATIONS OF KINETICS OF ACTIVATED SLUDGE TO PROCESS DESIGN

W. W. ECKENFELDER

Manhattan College

WHEN an organic waste is contacted with biological sludge, B.O.D. is removed by several mechanisms. Suspended and finely divided solids are removed by adsorption and coagulation. A portion of the soluble organic matter is initially removed by absorption and may be stored in the cell as a reserve food source. Additional dissolved organic matter is progressively removed during the aeration period resulting in the synthesis of sludge and the production of carbon dioxide and water. The rate of B.O.D. removal depends primarily upon the concentration of B.O.D. to be removed and the concentration of sludge solids. The reactions involved in the removal of B.O.D. from solution during bio-oxidation can be interpreted as a three-phase process (1):

(a) A high initial rate of removal of B.O.D. on contact of a waste with a biologically active sludge, a portion of which may be stored in the cell as a reserve food source.

(b) Removal of B.O.D. in direct proportion to biological sludge growth.

(c) Oxidation of biological cellular material through endogenous respiration.

VOL. 5 1961

B.O.D. REMOVAL RELATIONSHIPS

B.O.D. removal is intimately related to sludge growth. At high concentrations of B.O.D., the rate of sludge growth, and hence the B.O.D. removal rate is independent of B.O.D. concentration. Below a limiting B.O.D. level, the rate of sludge growth and the B.O.D. removal rate becomes concentration dependent. Relationships between sludge growth and B.O.D. removal have been developed for several waste oxidation processes (1–4) and are discussed by McCabe.*

At low B.O.D. levels, the removal of a single substrate from solution can usually be shown to follow the relationship:

$$\frac{dL}{dt} = K_1 L. \qquad (1a)$$

Equation (1a) can be generalized for a wide range of microbial solids concentrations:

$$\frac{dL}{dt} = K_1' S_a L. \qquad (1b)$$

* McCABE, B. J., this volume, p. 266

Garrett and Sawyer (3) have shown that at low B.O.D. levels, the sludge growth rate coefficient K is directly proportional to the concentration of B.O.D. remaining.

$$L = CK. \tag{2}$$

It can be shown that equation (2) is equivalent to equation (1b) under concentration limiting conditions.

It has been found in many biological oxidation systems that there is a very high initial rate of B.O.D. removal immediately on contact of waste with sludge, followed by a slower rate of removal. The magnitude of this high initial rate depends on the nature of the waste and the characteristics of the sludge. In many cases this initial rate of removal will exceed the maximum rate of sludge growth, resulting in a storage of B.O.D. in the microbial cell (5). This stored B.O.D. is subsequently oxidized over several hours aeration of the sludge. Eckenfelder (4) has shown that this high initial rate of B.O.D. removal can frequently be formulated as an exponential function of sludge solids over a short time interval. In applying this relationship, only that portion of the B.O.D. which is rapidly removed is considered.

In addition to a high initial removal rate, the heterogeneous nature of most waste mixtures results in a complex removal reaction. The various waste constituents are subject to varying reaction rates, resulting in a decreasing overall rate as the various components are oxidized. In some cases, a small fraction of the B.O.D. does not appear to be removed even after long aeration periods. Weston *et al.* (6) has attributed this phenomenon to an equilibrium between B.O.D. removal from solution and the release back to solution of the end products of respiration and auto-oxidation of cellular substance. In cases where this is observed it is frequently necessary to subtract this non-removable B.O.D. from the applied loading to formulate the rate equations.

It is possible to approximate the progress of oxidation of sewage and waste mixtures by modifying Equation (1b) to a composite exponential, which will consider all the various removal rates:

$$\sum_{1}^{n} \frac{dL}{dt} = [K'_a S_a L_1 + K'_b S_a L_2 + \cdots K'_n S_a L_n], \tag{3a}$$

which integrates to

$$\sum_{1}^{n} \frac{L_0}{L_0} = \exp(-K'_a S_a t) + \exp(-K'_b S\ t) \ldots + \exp(-K'_n S_a t), \tag{3b}$$

in which

$$L_0 = L_{0,n} + L_{0,1} + \ldots L_{0,2}$$

and

$$L = L_1 + L_2 + \ldots L_n$$

In equation (3) the high initial removal rate is reflected in the first term and the decreasing removal rate of the remaining waste constituents in the subsequent terms

of the equation. It has been observed that some B.O.D. is removed after 1–3 min. contact with activated sludge. Equation (3) will only approximate this immediate removal rate. Evaluation of data from several waste oxidation systems have shown that only three terms usually need to be considered. It is apparent from an examination of equation (3) that a high fraction of readily removable B.O.D., L_1 and a high initial removal rate, K'_a will result in high B.O.D. removals in short aeration periods. The form of equation (3) was used by Theriault and McNamee (7) to eva-

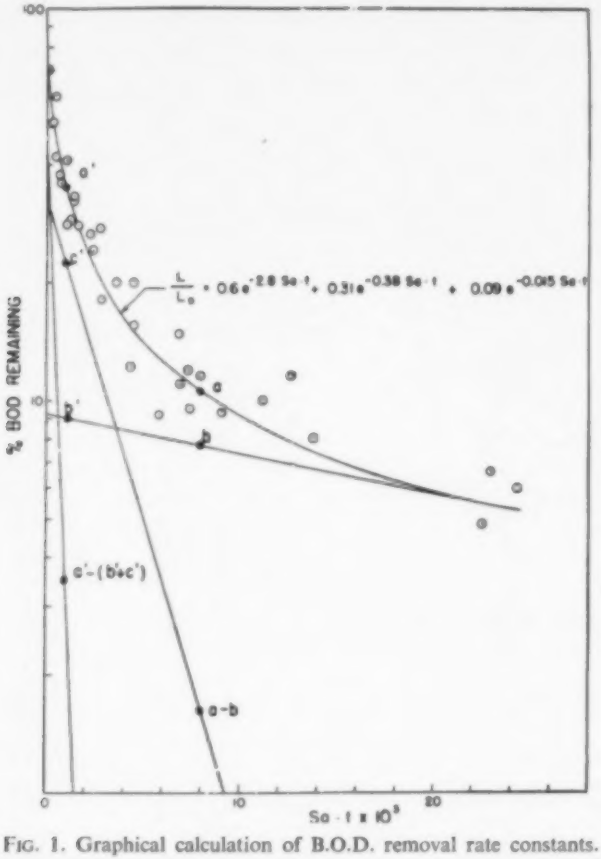


FIG. 1. Graphical calculation of B.O.D. removal rate constants.

luate the oxidation of channel sludges, by Moore (8) to evaluate the change in rate of oxygen utilization in activated sludge aeration, by Streeter (9) in the evaluation of coliform death rates and by Gameson and Wheatland (10) in the evaluation of the B.O.D. reaction.

Data from several waste oxidation systems were evaluated according to equation (3). The various rate constants and their associated B.O.D. concentrations were evaluated by the graphical method prescribed by Moore (8). The logarithm of the fraction of B.O.D. remaining (L/L_0) is plotted against the product of the mixed

VOL. 5 1961

liquor suspended solids and the aeration period ($S_a \cdot t$). In most waste oxidation systems this will yield a curve which initially drops very rapidly followed by a change in slope to a low linear rate at high values of $S_a \cdot t$. The various rate constants and their associated B.O.D. fractions are determined as follows: a straight line is drawn tangent to the lower end of the curve. The values of K_c' and L_3 are determined from the slope of this line and its intercept. The difference in B.O.D. remaining between this line and the removal curve is measured at suitable intervals and plotted on the same graph. These differences will be represented by a second straight line with a rate constant K_b' and an intercept L_2. At low values of $S_a \cdot t$ the curve will deviate from this second plotted line. A third rate constant is then determined by plotting the difference between the sum of the ordinates of the two removal lines from the ordinate of the curve. This plots a third straight line, the slope of which is K_a' and the intercept L_1. The sum of $L_1/L_0 + L_2/L_0 + L_3/L_0$ should equal 100 per cent. A typical calculation is shown in Fig. 1.

Table 1 summarizes the rate constants and their associated removable fractions for several waste oxidation systems.

TABLE 1

Waste	K_a'	L_1	K_b'	L_2	K_c'	L_3	Ref.
Domestic sewage and textile mill	4·35	0·28	0·95	0·55	0·061	0·175	(12)
Pulp and paper	1·73	0·17	0·50	0·29	0·051	0·33	
Pulping and bleaching	0·95	0·40	0·125	0·60	—	—	(11)
Sewage and paper mill*	4·20	0·40	0·57	0·40	0·057	0·18	
Settled sewage	2·8	0·60	0·38	0·31	0·015	0·09	(19)

* Based on filtered B.O.D. values.

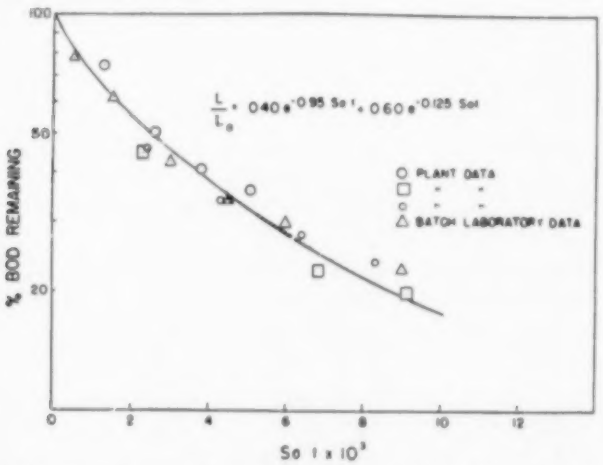


FIG. 2. B.O.D. removal characteristics in the treatment of a pulping and bleaching waste.

B.O.D. removal relationships from the bio-oxidation of a mixture of pulping and bleaching wastes at the West Virginia Pulp and Paper Co., Covington, Va. (11) during the period July–November, 1958, are shown in Fig. 2. The B.O.D. of the waste varied from 175 to 240 ppm. The aeration tanks in the waste treatment plant are divided into four compartments, each of which approaches a state of complete mixing. Samples were collected at the end of each compartment as shown in Fig. 2. For comparison results from a series of batch oxidation studies are also shown in Fig. 2. It will be observed from Table 1 that only two rate constants were found for this oxidation. It is probable that had the oxidation been extended further in time, a third, low rate constant would be developed. B.O.D. removal from a mixture of domestic sewage and a textile mill waste with an initial B.O.D. of 200–400 ppm after 4–5 hr sludge stabilization is shown in Fig. 3 (12). The plant performance

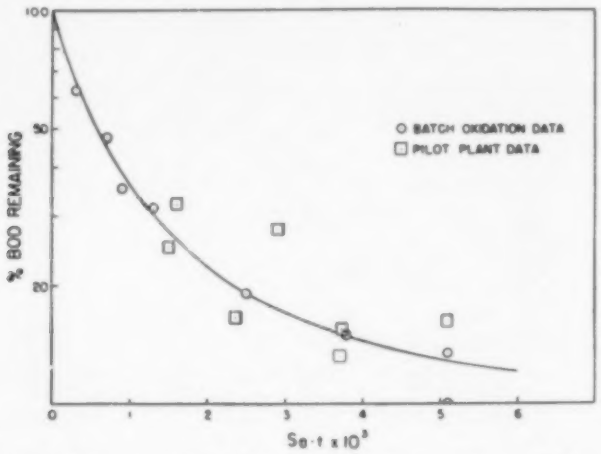


FIG. 3. B.O.D. removal characteristics in the treatment of domestic sewage and textile mill waste.

points represent monthly averages in which the aeration detention period varied from 0·5 to 1·4 hr and the mixed liquor suspended solids from 2100 to 4500 ppm. Batch oxidation results over a range of mixed liquor solids of from 1200 to 5400 ppm are also shown in this figure.

Several other mathematical relationships have been developed to consider the overall reaction rate K_1 to decrease with concentration or time. Fair and Geyer (13) have employed a generalized expression in which the reaction rate K_1 decreases with a decreasing availability of the substrate.

$$-\frac{dL}{dt} = K_1\left(\frac{L}{L_0}\right)^n L. \tag{4}$$

In equation (4) a decreasing reaction rate is reflected by the exponent n. When $n = 0$, equation (4) reduces to equation (1a). The removal of B.O.D. from a pulp and paper mill waste (initial B.O.D. 180–250 ppm) during a batch oxidation and

VOL. 5 1961

from a homogeneously mixed pilot plant is shown in Fig. 4. The line determined according to the integrated form of equation (4) is shown on the figure.

Fair and Moore (14) have employed a form of equation in which the reaction rate K_1 decreases with time.

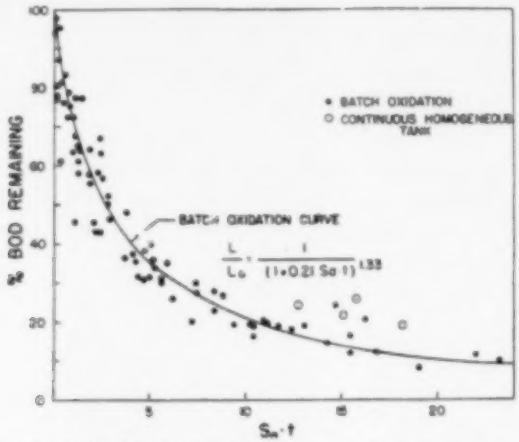


FIG. 4. B.O.D. removal relationship for batch oxidation and continuous treatment of a pulp and paper mill waste.

$$-dL/dt = \frac{K_1}{(1+mt)} L. \tag{5}$$

In cases where the initial removal rate is low, biological oxidation data can frequently be expressed by equations (4) and (5). High initial removal rates will cause a deviation from the equation at low values of $S_a \cdot t$.

FACTORS AFFECTING B.O.D. REMOVAL RATES

The high initial rate of removal of B.O.D. is affected both by the nature of the waste and by the condition of the sludge.

The variable nature of most industrial wastes results in a variation in removal rate as the composition of the waste changes. Table 2 shows the removal rate constants according to equation (3) for several mixtures of pulp and paper mill effluents. These data were obtained from a series of batch oxidation studies conducted simultaneously with the same concentration of activated sludge (3000 ppm).

TABLE 2*

Waste mixture	K_a	L_1	K_b	L_4
80% pulping–20% bleaching	1·7	0·73	0·11	0·27
60% pulping–40% bleaching	1·5	0·69	0·08	0·31
40% pulping–60% bleaching	1·6	0·68	0·03	0·32

* Data from the West Virginia Pulp and Paper Co., Covington, Va.

The magnitude of the initial removal is dependent on the condition of the sludge when it is contacted with waste. If the sludge has had an insufficient period of aeration prior to contact with waste, previously stored, B.O.D. will not be completely metabolized and the initial removal will be reduced. Results reported by the Water Pollution Research Laboratory (15) showed that ½ hr aeration after contact of sewage with sludge resulted in a progressive loss of clarifying power while 2 hr aeration maintained a continued high degree of clarification. On the other hand, excessively long sludge aeration periods result in extensive endogenous respiration and resulting loss in initial B.O.D. removal capacity of the sludge. Wuhrman (16) demonstrated a rapid reduction in the removal rate of lactose by activated sludge with sludge aeration periods up to 216 hr. The relation between sludge aeration or stabilization time and B.O.D. removal for various contact periods is shown in Fig 5. The data in Fig. 5 was obtained by aerating sludge with a mixture of domestic sewage

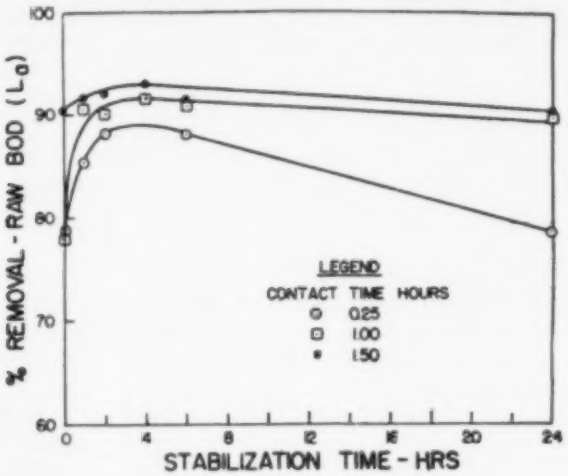


FIG. 5. Effect of stabilization on initial B.O.D. removal characteristics.

and textile mill waste for 15 min, separating the sludge, and aerating the sludge with waste for 0·25, 1·0 and 1·5 hr after the indicated stabilization periods. It is apparent from Fig. 5 and for a contact period of 0·25 hr and a mixed liquor suspended solids content of 2000 ppm a stabilization period of 2–6 hr is necessary to maintain the B.O.D. removal capacity of the sludge. Increasing the contact period permits some oxidation of the absorbed organics during the aeration period, thereby decreasing stabilization detention requirements. Ultimately, increasing the contact period will permit complete oxidation of the absorbed organics and eliminate the need for stabilization. This is then a conventional activated sludge process. Increasing the concentration of mixed liquor suspended solids will decrease stabilization requirements for the same B.O.D. loading, since the B.O.D. removed per unit sludge mass will be reduced. Under these conditions less time is required for the oxidation of the absorbed B.O.D. The short aeration period for high removal levels

VOL. 5 1961

without sludge stabilization reported by v. d. Emde (17) are made possible by the very high mixed liquor solids levels and consequent low B.O.D. removal per unit sludge mass.

OXYGEN UTILIZATION

During the B.O.D. removal process, oxygen is consumed to obtain energy for growth and for endogenous respiration. Relationships have been developed to define the oxygen requirements for various waste oxidation systems and for various loading conditions (18). The oxygen utilization rate can be related to the parameters of sludge growth by the relationship:

$$\frac{dO_2}{dt} = \left(\frac{a'}{a} K + b s\right). \tag{6a}$$

During the log growth phase when B.O.D. concentration does not limit the rate of reaction, equation (6a) reduces to:

$$k_r = C. \tag{6b}$$

During the declining growth phase K progressively decreases (equation 2) and the unit uptake rate k_r decreases and approaches the endogenous rate b. For this case equation (6a) can be combined with equation (2) to yield the relationship

$$k_r = C'L + b. \tag{6c}$$

It should be noted, however, that the specific uptake rate will depend on the history and acclimatization of the sludge. Hence an actively growing sludge will exhibit a more rapid response to B.O.D. loading than will an advanced endogenous sludge. This is illustrated in Figs. 6(a) and 6(b) which shows the unit

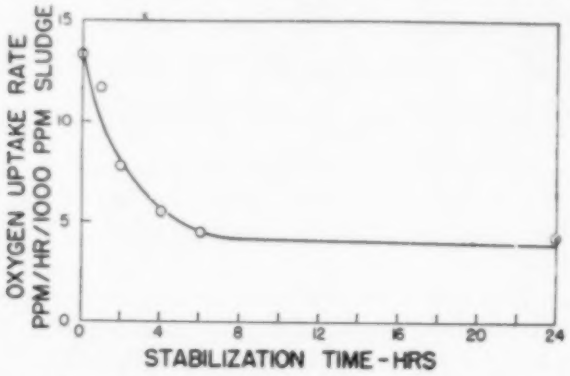


FIG. 6(a). Variation in unit oxygen uptake rate with stabilization time.

uptake rate of sludge mixed with sewage after various periods of sludge stabilization. The uptake rate after very short stabilization periods is high since previously removed B.O.D. is being oxidized in addition to the new organic loading. After long sludge aeration periods, the sludge responds slowly to the applied organic loading.

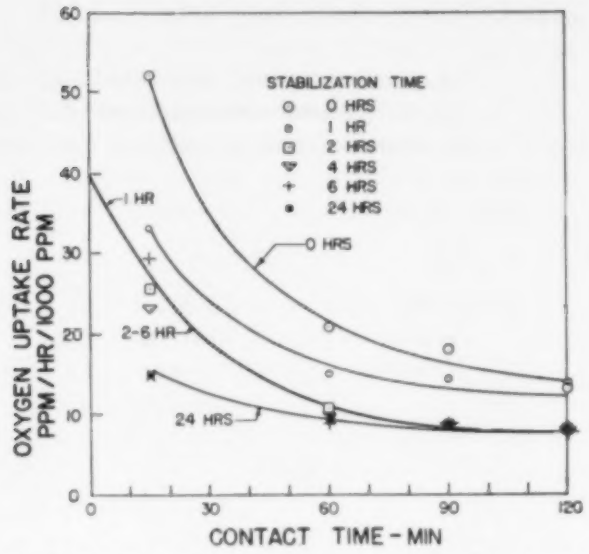


FIG. 6(b). Variation of unit oxygen uptake rate with contact time for various stabilization times.

VOL. 5 1961

APPLICATION TO BIOLOGICAL OXIDATION PROCESSES

A general schematic of the activated sludge process at low B.O.D. levels is shown in Fig. 7. When the initial removal rate and its associated removable fraction, K_a and L_1 are large, high removals can be attained in short aeration contact periods, resulting in a storage of some B.O.D. in the sludge. This B.O.D. is subsequently oxidized during a sludge aeration or stabilization period. Application of this concept

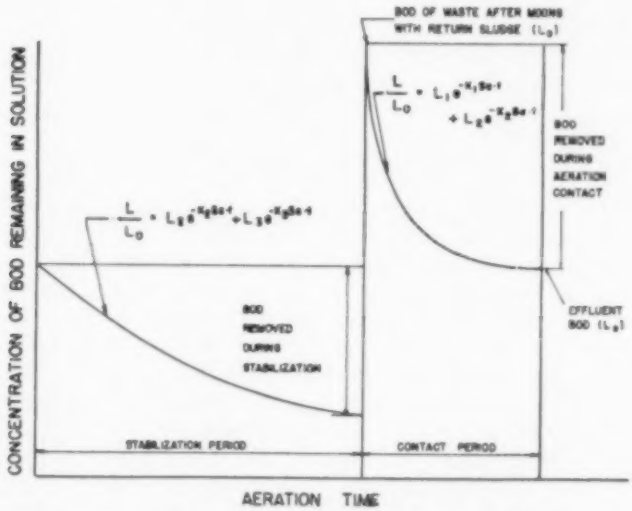

BOD OF WASTE AFTER MIXING WITH RETURN SLUDGE (L_0)
$\frac{L}{L_0} = L_1 e^{-K_1 5 t} + L_2 e^{-K_2 5 t}$
BOD REMOVED DURING AERATION CONTACT
$\frac{L}{L_0} = L_1 e^{-K_1 5 t} + L_2 e^{-K_2 5 t}$
BOD REMOVED DURING STABILIZATION
EFFLUENT BOD (L_e)
CONCENTRATION OF BOD REMAINING IN SOLUTION
STABILIZATION PERIOD
CONTACT PERIOD
AERATION TIME

FIG. 7. Schematic representation of the activated sludge process.

is known as the contact stabilization process. Increased removals can be obtained by extending the contact period and reducing the sludge stabilization period (see Fig. 5). When the initial removal rate is low, the mixed liquor suspended solids high, or the aeration contact period extended beyond approximately 2 hr, the need for sludge stabilization is eliminated. This is typical of conventional activated sludge operations. The selection of type of process and mode of operation will depend primarily on the relative magnitude of the removal rate constants.

SUMMARY AND CONCLUSIONS

Formulations have been developed to express the rate of B.O.D. removal in various types of wastes and at various loading levels. Depending on the magnitude of the removal coefficients, various activated sludge process modifications can be employed for high degree treatment. The oxygen utilization rate which may be expected for various B.O.D. loading levels is expressed in terms of the parameters of bacterial growth and endogenous respiration.

NOMENCLATURE

a, fraction of B.O.D. synthesized to sludge.
a', fraction of B.O.D. oxidized.
b, endogenous respiration rate (hr^{-1}).
C, constant.
k, constant.
k_r, oxygen uptake rate, mg O_2/hr/g sludge.
K, sludge growth rate constant, hr^{-1}.
K_1, B.O.D. removal rate constant, hr^{-1}.
K'_a, K'_b, K'_c, B.O.D. removal rate constants per unit sludge mass.
L, B.O.D. concentration.
L_0, initial B.O.D. concentration at $t = 0$.
S_a, mixed liquor solids concentration, g/l.
t, time, hr.

REFERENCES

1. WESTON, R. F. and ECKENFELDER, W. W., Application of biological treatment to industrial wastes. *Sewage Industr. Wastes* **27**, 7, 802 (1955).
2. ECKENFELDER, W. W., JR. and MCCABE, B. J., Process design of biological oxidation systems for industrial waste treatment. *Proc. Symposium on the Treatment of Waste Waters, Univ. of Durham, King's College, Newcastle-upon-Tyne, England*. Pergamon Pub. Co. 1962.
3. GARRETT, T. M. and SAWYER, C. N., Kinetics of removal of soluble BOD by activated sludge. *Proc. 7th Ind. Waste Conf., Purdue University*, p. 51 (1952).
4. ECKENFELDER, W. W., JR., Kinetic relationships in the bio-oxidation of sewage and industrial wastes. *Proc. 15th Ind. Waste Conf., Purdue University* (1959).
5. PORGES, N., Biochemical oxidation of dairy Wastes—VII. Purification, oxidation, synthesis and storage. *Proc. 10th Ind. Waste Conf., Purdue University* p. 135 (1955).
6. WESTON, R. F., STACK, V. T. and SITMAN, W., Mass transfer relationships in biological treatment. Paper presented A. I. Ch. E., Atlanta, Ga. (1960).

7. THERIAULT, E. J. and MCNAMEE, P. H., Sludge-aeration experiments. *Industr. Engng. Chem.* **22**, 12, 1330 (1930).
8. MOORE, E. W., The oxygen demand of mixtures of activated sludge and sewage. *Sewage Wks. J.* 9, 1, 12 (1937).
9. STREETER, H. W., A formulation of bacterial changes occurring in polluted water. *Sewage Wks. J.* 6, 2, 208 (1934).
10. GAMESON, A. L. H. and WHEATLAND, A. B., The ultimate oxygen demand and course of oxidation of sewage effluents. *J. and Proc. Inst. Sewage Purif.* pt 2 (1958).
11. West Virginia Pulp and Paper Co., Covington, Va., Private communication.
12. Cone Mills, Inc., GREENSBORO, N. C., Private communication.
13. FAIR G. M. and GEYER J. C., *Water Supply and Waste Water Disposal*. John Wiley, New York (1954).
14. FAIR, G. M. and MOORE, E. W., Mathematical formulation of the course of digestion. *Sewage Wks. J.* **4**, 429 (1932).
15. Water Pollution Research, Dept. of Scientific and Industrial Research, London (1956).
16. WUHRMAN, K., *Biological Treatment of Sewage and Industrial Wastes* (edited by MCCABE, B. J. and ECKENFELDER, W. W.) vol. 1. Reinhold, New York (1957).
17. V. D. EMDE, W., Aspects of the high rate activated sludge process. This volume, p. 299.
18. ECKENFELDER, W. W. and O'CONNOR, D. J., Biological oxidation of organic wastes. *Proc. 10th Ind. Waste Conf., Purdue University* (1956).
19. RUCHHOFF, C. C., BUTTERFIELD, C. T., MCNAMEE, P. D. and WATTIE, E., Studies of sewage purification—IX *Sewage Wks. J.* **11**, 2, 195 (1939).

VOL. 5 1961

DISCUSSION

W. F. HOWLAND: To answer criticisms of the "composite exponential" type of equation we have found this same type of formula to follow Keefer's Baltimore data. My curves are shown on Fig. 3 of Ref. (5) of my paper (this volume, p. 248). I have shown that equations of this type—in particular with two exponential terms—result either from (a) assuming two fractions of oxidizable constituents each of which decomposes at a different rate of K value, or (b) by assuming that there is one component, A, initially which goes to B and requires oxygen in so decomposing and that this component B is oxidized to C requiring somewhat different amounts of oxygen and proceeding at a different K rate. This also is shown in Ref. (5).

AUTHORS: Keefer's Baltimore data has been evaluated employing a similar approach by this author. This data has been correlated employing multiple regression analysis. The resulting equation is

$$\%\text{B.O.D.}_{\text{remaining}} = \frac{100}{1 + 4{\cdot}25\, D^{0{\cdot}650}/Q^{0{\cdot}411}}$$

The coefficient of correlation obtained was 0·87.

K. WUHRMANN: The following remarks are intended as a critical consideration of two assertions on which the two previous papers, as well as many previous publications by other authors are based. These two assertions are:

(1) B.O.D. is a quantitative measure for the concentration of metabolic substrates, oxidizable by saprophytic organisms, and the change of B.O.D. in the supernatant of a bacterial suspension is quantitatively correlated with different metabolic reactions in the organisms (i. e. growth, proliferation, katabolic and anabolic activities, etc).

(2) Organic, oxidizable material in wastes is eliminated by organisms from solution according to a first order reaction.

Significance of the term B.O.D.: B.O.D. is a quantity of oxygen, disappearing from a so-

lution by the activity of micro-organisms, i. e. by their respiration. This oxygen consumption is the final step in a long chain of reactions, the purpose of which is the gain of energy in a utilizable form to the cells. The first step in this reaction chain is generally a dehydrogenation of suitable organic or inorganic molecules which are either resorbed from the external medium or which constitute secondary products of metabolic steps.

It is rather difficult to conclude from the quantity of oxygen consumed by cells, as to which quantity of substrate molecules have been taken up by them from solution, because two fundamental details are completely ignored, namely:

(a) the proportion of molecules resorbed by the cells, being introduced into the energy yielding respiration chain (i. e. causing finally an oxygen consumption); and

(b) the distribution of the gained energy to the innumerable endergonic cell activities (for instance growth, i. e. increase of weight and size of the cells; or proliferation, i. e. formation of new cells).

The correlation of oxygen consumption and substrate uptake may lead to more or less reproducible results with pure cultures, strictly standardized metabolic conditions and well defined substrates. It is impossible, however, to obtain any conclusive relation between simple metabolic parameters (increase in weight, oxygen consumption, etc.) and the kinetics of the concentration decrease of organic compounds in complex substrates which are oxidized by a mixture of organisms with different metabolic activities and highly diverging nutritional requirements.

It is therefore a fundamentally wrong concept to use the B.O D. in calculations as if it were a quantity of metabolic substrate, underlying anabolic or katabolic reactions within cells. A much more careful use of the term B.O.D. is necessary in connection with physico-chemical and biochemical considerations in biological waste treatment. Expressions such as "B.O.D. is adsorbed", "storage of B.O.D. within the cells", "oxidation of the B.O.D.", etc., which are quite frequently found in literature, should be eliminated.

It has been frequently demonstrated that the concentration decrease of a pure compound in suspensions of microorganisms is of zero order, e. g. penicillin producing organisms,*† trickling filter slimes and pure cultures of trickling filter organisms,‡ activated sludge. § ** †† Winzler‡‡ and recent authors have further established that the oxygen consumption of yeasts and bacteria is of zero order within a wide concentration range in solution (lower limit of linearity: the critical concentration, order of magnitude: $< 0{\cdot}1$ to $0{\cdot}5$ ppm). All these linear reactions are due to the fact that the uptake of molecules from the surrounding medium by the cell depends largely on the number of carrier molecules and acceptor-loci (enzymes) in the cell surface and protoplasm for the particles to be resorbed. In view of the generally rather low substrate concentrations required for maximum saturation velocities of the known receptors (i. e. the rate of formation of the receptor-substrate complex), the transport mechanism for the substrate molecules from the external solution to the receptors is more or less the only factor determining the kinetics of concentration decrease in the medium. This mechanism being mostly diffusion, it is evident that the concentration decrease follows a zero-order reaction. A change of reaction characteristics takes place as soon as the external concentration has become so small that diffusion rates are less than fixation rates at the cell receptors (critical concentration). These critical concentrations are mostly beyond the detection limits of usual quantitative analytical procedures for organic substrates.

It is certainly a justified request by engineers to get correlations of plant performance and operational variables, such correlations being indispensable tools for the dimensioning of treatment plants. There is neither an objection against the use of empirical parameters like B.O.D. for this purpose nor against the formulation of such correlations by mathematical means. It is, however, not possible and very misleading to use these empirical relationships for the deduction of reaction kinetics of the metabolic steps involved in the biological purification of wastes by micro-organisms.

REFERENCES

* Brown, W. E. and Peterson, W. H., *Industr. Engng. Chem.* **42**, 1769, 1823 (1950).

† Bartholomew, W. H., Karow, E. O., Sfat, M. R. and Wilhelm, R. H., *Industr. Engng. Chem.* **42**, 1810 (1950).

‡ Water Pollution Research Laboratory, England, Annual Rep. 1955. Her Majesty's Stationery Office, London.

§ Wuhrmann, K., *Biological Treatment of Sewage and Industrial Wastes* vol. 1, p. 49. Reinhold, New York (1956).

** Wuhrmann, K. Beust, F. V. and Ghose, T. K., *Schweiz. Z. Hydrol.* **20**, 284 (1958).

†† Wuhrmann, K. and Beust, F. V., *Schweiz. Z. Hydrol.* **20**, 311 (1958).

‡‡ Winzler, R. J., *J. Cell. Comp. Physiol.* **17**, 263 (1941).

Authors: (1) The disadvantages of both B.O.D. and C.O.D. as parameters of substrate concentration are well appreciated. They do, however, provide convenient, if crude, measures of the concentration of oxidizable organic matter in complex waste substrates when determination of individual substrate components is impossible. It is in this sense that the various authors referred to use the terms.

(2) Most batch oxidations begun at relatively high ratios of food to organisms exhibit sigmoidal curves for biological growth, B.O.D. reduction, and non-specific substrate removal. The linear portions of these curves (*e–f* in Fig. 1, p. 265) can be considered to represent zero-order reactions but within much more restricted limits than those obtained with pure substrates. Dr. Wuhrmann defines the lower critical substrate concentration which limits linearity as that when diffusion rates are smaller than enzymatic fixation rates. Above this critical limit simple substrate removals were found to be linear with time and proportional to the concentration of organisms. Non-linear removals were observed for complex substrates such as peptone, or when either B.O.D. or organic nitrogen were employed as parameters; these non-linear removals were interpreted as the summation of several linear removal rates. In the previous two papers the overall non-linear B.O.D. removals were formulated as either first-order or retardant equations during the postulated Phase II.

Since substrate removals are proportional to the concentration of microorganisms, linearity is affected by the growth rate during batch oxidations. This growth effect is masked when high initial ratios of organisms to substrate are used. With initially low concentrations of organisms, removals of pure substrates and of B.O.D. follow the equations presented for Phase I if adsorbed material is taken into account.

Formulations such as those presented in these papers and by others* are needed to follow or predict the results observed during sludge growth, B.O.D. removal and oxygen utilization during biological oxidations. They cannot, of course, substitute for fundamental research.

* Busch, A. W. and Myrick, N., *Proceedings, 15th Ind. Waste Conf., Purdue University* (1960).

VOL. 5 1961

DEVELOPMENTS IN ACTIVATED SLUDGE TREATMENT IN THE NETHERLANDS

A. PASVEER

Research Institute of Public Health Engineering, Holland

REPORTS of previous work published in the United States have dealt with the theory of oxygenation (Pasveer, 1953a) and an investigation conducted on the Kessener

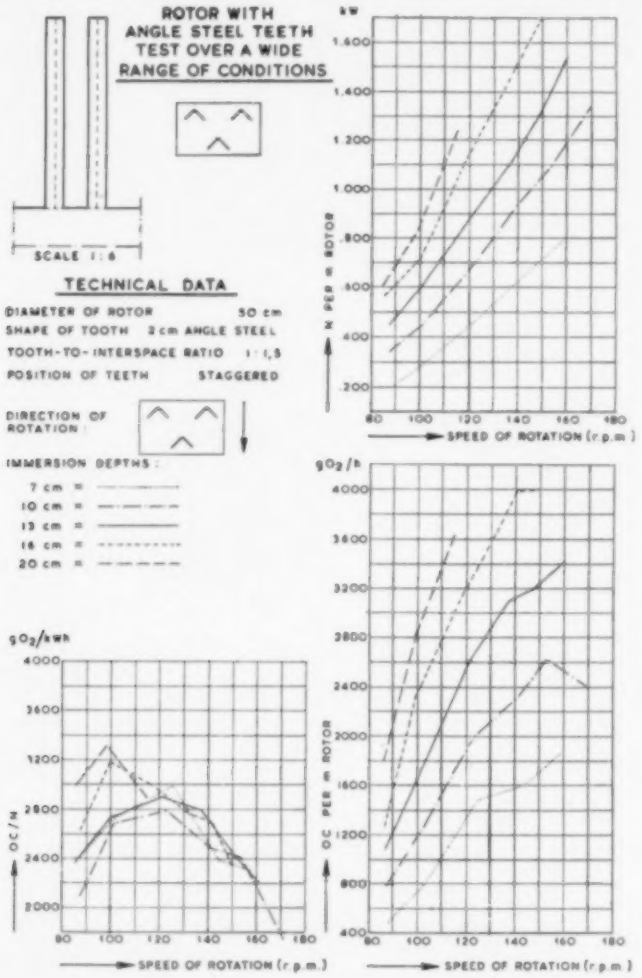


FIG. 3. Oxygenation capacity and energy consumption of an angle-iron rotor at different depths of immersion and different speeds of rotation.

VOL. 5 1961

brush aeration system (Pasveer, 1953b). In the course of the latter research it was found that the oxygenation of the water in this system takes place entirely in the near vicinity of the brush. The physical process of oxygenation can be carried out

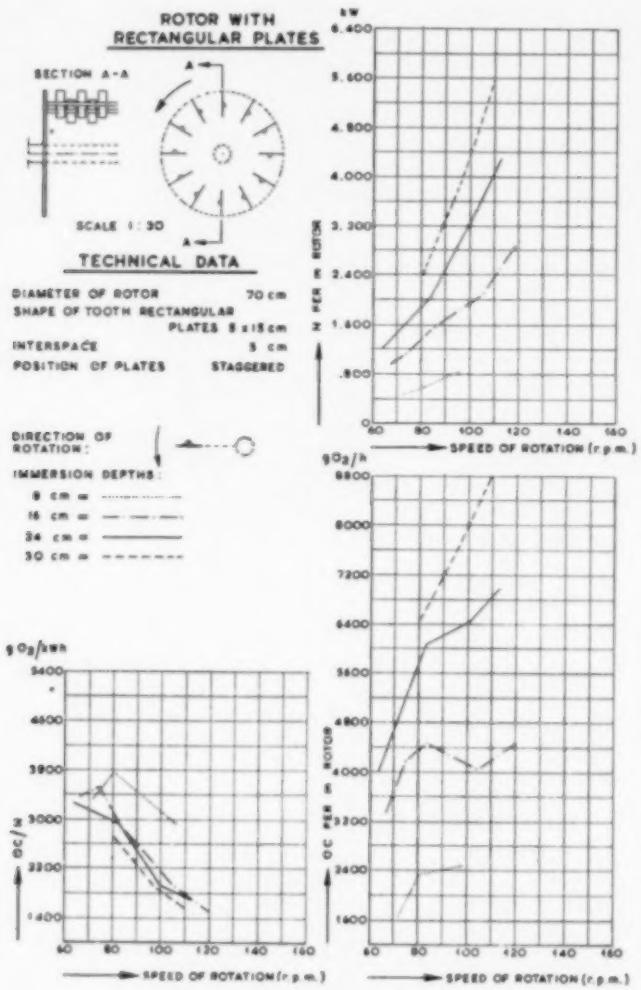


FIG. 4. Oxygenation capacity and energy consumption of a cage rotor at different depths of immersion and different speeds of rotation.

just as satisfactorily in a small tank as in a large one. This result stimulated research in different directions. On the one hand, it provided a research tool in the search for a method of improved oxygenation, and activities in this direction led to the development of new types of aeration rotors. On the other hand, it became possible to conduct experiments to ascertain the rate of oxygen-consumption in the activated sludge process during very intense aeration (Pasveer, 1954).

VOL.
5
1961

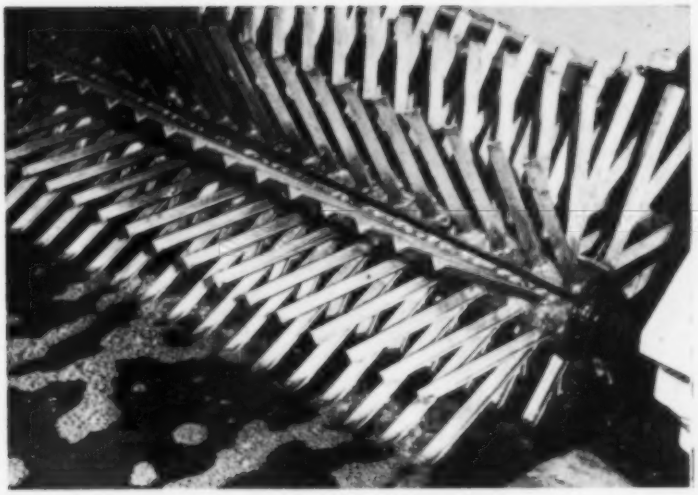


FIG. 1. Angle-iron aeration rotor.

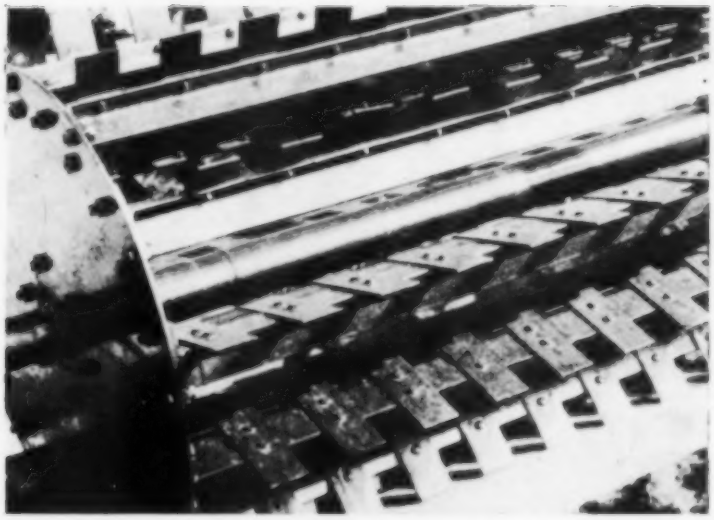


FIG. 2. Cage aeration rotor.

A report of intensive research on the capacity and efficiency of rotating brushes conducted by Baars and Muskat (1959) has recently been published. Figs. 1 and 2, taken from that paper, show two types of aeration rotors: a so-called angle-iron aeration rotor which may be used in the normal activated-sludge process, and a so-called cage rotor designed for use in the oxidation ditch, particulars of which are given later in this paper. The yield per meter of brush for these aeration rotors is from two to four times as great as that of the conventional Kessener brushes, while the efficiency (yield in grams of oxygen per kW) is also somewhat higher (Figs. 3 and 4).

OXYGENATION CAPACITY

The purification capacity of an activated sludge plant is closely related to the rate of oxygenation. With this in mind, Kessener and Ribbius (1934) introduced the concept of "oxygenation capacity" (O. C.) of an aeration tank, defined as the rate of oxygenation in (distilled) water under the fixed conditions of 10°C, 760 mm Hg and zero oxygen content. The oxygenation capacity of an aeration plant may be expressed in g O_2 per hr per m^3 of aeration tank (or oz per hr per 1000 ft^3) and may be determined in a simple manner (Pasveer, 1953b). The application of this concept allows comparison between results found under varying conditions (*ibid*). Its determination is essential for judging the capacity and efficiency of an aeration plant

VOL. 5 1961

RESEARCH ON THE RATE OF OXIDATION IN AN ACTIVATED SLUDGE TANK

It is known that, barring exceptional conditions, a given quantity of sewage can be completely purified in the activated sludge process if a sufficient amount of oxygen is supplied and if provision is made for a sufficiently long aeration period. It is also known that by increasing the supply of oxygen, it is possible to purify a larger quantity of sewage. The development of the activated sludge process gives a clear illustration of this. The load considered normal for an activated sludge plant with complete purification shows an increasing trend which has been particularly marked during the last 10 years, thanks to the application of more intensive aeration.

This increase in the loading capacity of an aeration tank is accompanied by a shortening of the retention time. Where formerly an aeration period of 10–12 hr or longer was deemed necessary, it is now found that with more intensive aeration, an aeration period of 5–6 hr is sufficient; or, to use a better standard, where formerly a load of 500 g B.O.D. per m^3 of aeration tank per 24 hr was considered normal, this load has gradually been doubled.

An important problem still awaiting solution is: how far is it possible to go in this direction? Or, in other words, to what extent is the oxygenation capacity the limiting factor that determines the purification capacity of a plant? Should it prove possible by efficient means to double or further multiply the oxygenation capacity in an aeration plant, and should it be found that the purification capacity of the plant can be increased in proportion, a much smaller tank volume would serve the

purpose and in this way, a great saving in construction costs would be effected. Operation with a rotating brush in a small aeration tank allows the rate of oxygenation per unit volume to be several times that obtained in a tank of usual size. Use of this very convenient research tool allows us to ascertain whether, at very high rates the rate of oxygen consumption also is much higher. Experiments previously reported (Pasveer, 1954) were conducted in a small treatment plant, using activated sludge with brush aeration.

Results of these experiments showed that by increasing the oxygenation capacity (O. C.), it is possible to effect complete purification of much larger amounts of sewage per m^3 of aeration tank volume than had formerly been considered possible.

From a scientific point of view it was of interest to investigate whether the complete purification of still higher loading could be achieved. This experiments were carried out with the aid of a small laboratory activated sludge apparatus (Pasveer, 1955). It proved possible with very intense aeration (O. C. between 3000 and 4000 $g/m^3/hr$) to purify a synthetic sewage (400 ppm B.O.D.*) to the extent of 93 per cent with a load of 60,000 $g/m^3/24$ hr and an aeration period of only a few minutes (20°C). Furthermore, at the same temperature, a cotton-filtered sterilized domestic sewage (220 ppm B.O.D.) was purified to the extent of 96 per cent with a load of 24,000 $g/m^3/24$ hr and 92 per cent (10°C).

These results showed that up to very high oxygenation capacities (about fifty times as high as in practice) the oxygen comsumption by the activated sludge floc may be as intense as the oxygen supply.

Application in practice

Kehr and Schmidt Bregas (1957), von der Emde (1957), McNicholas (1957) report on the application of brush aeration in large plants and in pilot plants using only a small tank volume in order to obtain a high aeration intensity.

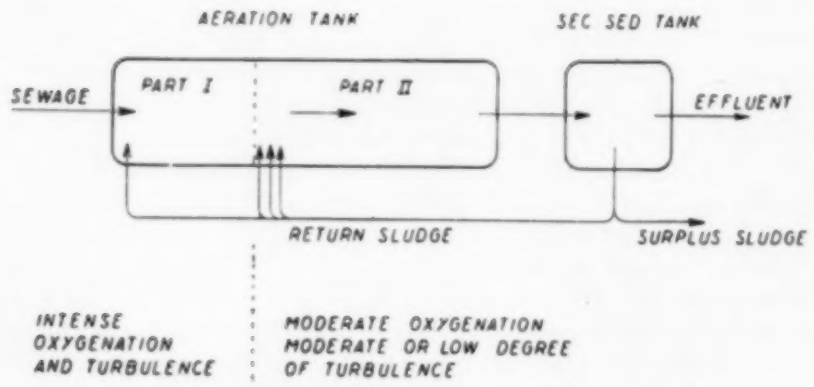


FIG. 5. Scheme of experimental activated-sludge plant.

* All B.O.D. values are for 5 days and 20°C.

In these applications a high purification capacity with a high degree of purification (i.e. a clear effluent having 10–20 ppm B.O.D.) is associated with a high expenditure of energy. The reason may be that in research to date no attention has been paid to the different demands of oxygenation and of turbulence in the first and second part of the aeration tank. Further experiments, for instance with the scheme of Fig. 5 are required to learn the best conditions of oxygen supply, turbulence and quantity of floc which are required, especially in the second part of the aeration tank to achieve a clear effluent in a short time with a low expenditure of energy. (For more details see Pasveer, 1960).

RESEARCH ON SIMPLIFIED SEWAGE PURIFICATION FOR COMPARATIVELY SMALL AMOUNTS OF SEWAGE AND INDUSTRIAL WASTES

The problem

The purification of waste water from villages, small residential communities, small industrial works, dairies, etc., is a great problem. The costs of building and running a small purification plant are so high that in a large number of cases, the plants needed cannot be built. In the last few years, the. Institute of Public Health Engineering T.N.O. has been engaged in research to find a method whereby small amounts of sewage can be purified at such a cost that the method may find general adoption.

VOL. 5 1961

Simplified method of sewage purification

This research has led to a greatly simplified purification technique. The method consists in the main of a single process, viz. that of oxygenation in an oxygenation tank of the simplest possible construction, usually an earthen ditch. The sewage

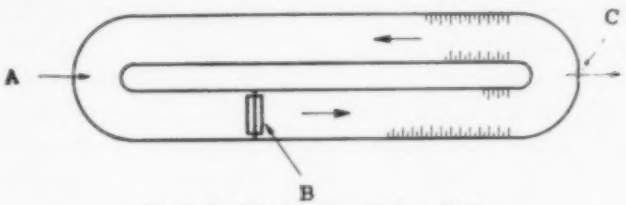


FIG. 6. Basic form of oxidation ditch.

is not subjected to any preliminary treatment. By intensive aeration it is possible to purify the sewage fully and also to mineralize the fresh sludge carried by the sewage together with the sludge formed in the purification process to such an extent that the surplus sludge can be dried without thereby causing any objectionable odors. A sludge-digestion tank is therefore unnecessary. By selecting a suitable working method, it is furthermore possible to avoid building a secondary sedimentation tank with a return system.

The "basic form" of plant is a ring-shaped circuit or ditch (Fig. 6). Mounted in this ditch is an aeration rotor *B* which provides for oxygenation as well as circu-

lation. In this way the mixture of sewage and floc in the circuit is made to pass the aeration rotor regularly at brief intervals during which it becomes aerated.

For domestic sewage the capacity of the aeration tank is 300 l. per population equivalent.* The raw sewage entering at *A* becomes diluted with the large amount of purified sewage present in the ditch. After complete biological purification the effluent is withdrawn in such a manner that all the floc (sludge) is retained in the plant and a clear effluent is drawn off at *C*. The quantity of suspended matter in the plant therefore constantly increases.

Owing to the large volume of the aeration tank and to the fact that the suspended matter in the aeration tank is kept fairly high, the total amount of floc in the plant is from ten to thirty times as great as that in a conventional activated sludge plant. The rate of B.O.D. loading is correspondingly lower. For a conventional type of activated sludge plant this loading rate is, for instance, 700 g B.O.D./kg floc/24 hr, whereas in the large aeration tank, the loading rate in only 50 g B.O.D./kg floc/24 hr. It is evident that if a sufficient amount of oxygen is provided at this very low loading rate, the floc is in an advanced stage of mineralization.

As soon as there is a sufficiently high sludge content in the plant, the operation is changed over so as to effect regular removal of a quantity of surplus sludge to maintain a constant suspended solids content in the circuit.

From the above discussion about the principles underlying the simplified system of purification, it is possible to draw the following conclusions;

(1) With the simplified process the energy required for oxygenation will be greater than is the case in a conventional activated sludge plant. This is obvious when we consider that it is necessary to satisfy the B.O.D. not only of a presettled sewage but also of the fresh sludge arriving in the plant; moreover, the total quantity of sludge in the plant must be brought to an advanced stage of mineralization. However, the cost of the necessary additional consumption of energy is only a small proportion of the saving in capital costs.

(2) Every class of sewage or waste susceptible to purification by biological oxidation, e.g. by the use of trickling filters or by the activated process, can undoubtedly also be biologically purified to the full extent in the simplified process.

(3) The very large amount of *suspended matter* renders the process insensitive to peak B.O.D. loads. Thanks to the *large volume* of the plant, it is less sensitive to sudden increases in the supply of toxic constituents.

(4) It is to be anticipated that the purifying capacity of the plant will be less susceptible to the influence of low temperature than the conventional activated sludge plant which already is not sensitive in this respect.

Experiments and use in practice

On the basis of the foregoing principle experiments have been made with a plant for 400 inhabitants (Fig. 7).

* One population equivalent = 54 g B.O.D.

VOL.
5
1961

FIG. 7. Oxidation ditch at Voorschoten (400 inhabitants).

In this plant a complete purification of the sewage is obtained with partial nitrification. The sludge at the same time is mineralized to such an extent that it can be dried on drying beds without causing bad odours. Details, results and data on the development of the method are included in Pasveer (1960).

Since the results with this oxidation ditch were first published four years ago, this simplified method of waste purification has found application in a number of cases. Fifty oxidation ditches are already in operation in the countries of Western Europe, and an equal number is either already designed or under construction.

The simplified system is applicable not only to domestic sewage but also to the purification of organically polluted industrial wastes. A few oxidation ditches used for the purification of dairy wastes give excellent results.

Installation costs for the oxidation ditches built in the Netherlands up to now are between 20 and 50 guilders per inhabitant with an average of 30–35 guilders (10 guilders = $ 2·65). The working and maintenance costs consist mainly of power costs for aeration. The power required for this purpose does not exceed 18 kWh per population equivalent (54 g $B.O.D._5$) per year. From the foregoing it follows that the total costs per inhabitant per year do not come to more than 5–6 guilders (about $ 1·50). Thus, with the method described above, it is possible to purify the sewage of small residential communities and small industrial works at a cost of a few guilders per population equivalent per year which was formerly possible only in the case of larger plants.

VOL.
5
1961

REFERENCES

BAARS, J. K. and MUSKAT J., *Oxygenation of Water by Bladed Rotors*. Report No. 28 of the Research Institute for Public Health Engineering TNO, The Hague (1959).

VON DER EMDE, W., Beitrag zu Versuchen zur Abwasserreinigung mit belebtem Schlamm. *Veröffentlichungen des Institutes fur Siedlungswasserwirtschaft der Technischen Hochschule Hannover*. Eigenverlag des Institutes, Hannover (1957).

KEHR, D. and SCHMIDT BREGAS T., Die neue Kläranlage der Stadt Detmold. *Bauamt u. Gemeindebau* **30**, 240 (1957).

KESSENER, H. J. N. H. and RIBBIUS, F. J., Comparison of aeration systems for the activated sludge process. *Sewage Wks. J.* **6**, 423 (1934).

MCKINNEY, R. E., Biological flocculation. *Biological Treatment of Sewage and Industrial Wastes* (eited by MCCABE, J. and ECKENFELDER, W. W.) vol. 1, p. 88. Reinhold, (New York) (1957).

MCNICHOLAS, J., Report Rivers Department, City of Manchester for the year ended 31 March 1957.

PASVEER, A., Research on activated sludge—I. A study of the aeration of water. *Sewage Industr. Wastes* **25**, 1253 (1953a).

PASVEER, A., Research on activated sludge—II. Experiments with brush aeration. *Sewage Industr. Wastes* **25**, 1397 (1953b).

PASVEER, A., Research on activated sludge—IV. Purification with intense aeration. *Sewage Industr. Wastes* **26**, 149 (1954).

PASVEER, A., Research on activated sludge—V. Rate of biochemical oxidation. *Sewage Industr. Wastes* **27**, 783 (1955).

PASVEER, A., New developments in the application of Kessener brushes (aeration rotors) in the activated-sludge treatment of trade-waste waters. *Proc. Symp. on Treatment of Waste Waters* (edited by ISAACS, P.). Pergamon Press, London (1960).

ASPECTS OF THE HIGH RATE ACTIVATED SLUDGE PROCESS

W. VON DER EMDE

Department of Public Works, Hamburg, Germany

EXPERIMENTAL studies were conducted in two experimental plants to determine B.O.D. removal efficiencies by the high rate activated sludge process under various operating conditions. The first plant was constructed at Kassel and consisted of three units, each having an aeration tank volume of 708 ft³. The units were equipped with diffused aeration, Kessener brushes and the INKA aeration system, respectively. Each unit treated 0·13 mgd of primary settled domestic sewage with an average B.O.D. of 140 ppm. The second plant was constructed at the town of Detmold and consisted of primary sedimentation, activated sludge aeration and secondary sedimentation. The waste treated consisted of a mixture of domestic sewage and fibre board waste.

VOL. 5 1961

EFFECT OF HYDRAULIC LOADING

Fig. 1 shows the effect of hydraulic loading on removal efficiency. The theoretical detention period in the aeration tank (without considering return sludge) is recorded below the line and the hydraulic load R_h (Wuhrmann, 1954) above the line. The experiments (Table 1) at Manchester, England (McNicholas and Tench, 1959)

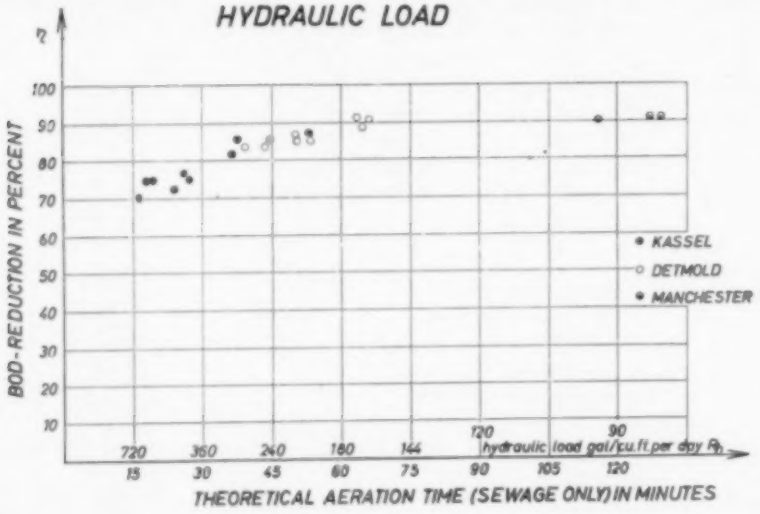


FIG. 1.

have shown that there is but a small difference in B.O.D. reduction (about 1 per cent) between a detention period of 2 hr and one of 8 hr. In each case the B.O.D. reduction was more than 90 per cent. The experiments of Pasveer (1954) at Klockenburg,

TABLE 1

Plant	T (min)	R_h	R_b	S.S. (ppm)	Index	$\frac{R_b}{R_{TS}}$	$\frac{R_N}{Rb'}$	B.O.D. infl. (ppm)	B.O.D. effl. (ppm)	red. (%)	Aeration system
Manchester (McNicholas and Tench, 1959)	500	21·8	37·5	—	—	—	0·25	199	16	92	Simplex-Syst. (H. J. Cone)
	470	23·2	38	—	—	—	0·25	194	15	92	Simplex-Syst. (H. J. Cone)
	335	32·3	59	4600	71	0·2	0·25	220	18	92	Simplex-Syst. (H. J. Cone)
	300	36·0	59	3600	61	0·25	0·25	195	15	92	Simplex-Syst. (H. J. Cone)
	130	82·5	132	3500	81	0·6	0·33	191	17	91	Simplex-Syst. (H. J. Cone)
	127	85	150	4900	63	0·5	0·23	212	19	91	Prototype
	350	31·5	53	—	—	—	0·28*	205	21	90	Kessener-Br.
	270	40	72	4200	75	0·27	0·37*	212	15	93	Kessener-Br.
	116	87	177	3100	86	0·9	0·45*	229	22	90	Pasveer-Unit
Detmold	65	167	225	7700	73	0·47	0·572†	161	17	89	Kessener-Br. (Pasveer-Unit)
	66	165	281	7900	65	0·57	0·450†	204	18	91	Kessener-Br. (Pasveer-Unit)
	63	172	494	7900	50	1·0	0·258	345	27	92	Kessener-Br. (Pasveer-Unit)
	50	218	306	10200	73	0·5	0·335	167	21	87	Kessener-Br. (Pasveer-Unit)
	50	214	431	5900	50	1·16	0·272	242	35	86	Kessener-Br. (Pasveer-Unit)
	53	202	644	8800	50	1·17	0·195	379	53	86	Kessener-Br. (Pasveer-Unit)
	43	253	312	11200	40	0·45	0·312	147	21	84	Kessener-Br. (Pasveer-Unit)
	44	247	455	8900	44	0·82	0·244	224	30	86	Kessener-Br. (Pasveer-Unit)
	39	277	325	8900	42	0·58	0·326	141	23	84	Kessener-Br. (Pasveer-Unit)
Kassel	53	202	238	4200	80	0·9	0·232	141	19	87	Diffused-Air (Brandol 60)
	36	300	350	3900	65	1·4	0·216	141	25	82	Diffused-Air (Brandol 60)
	37	291	206	4800	70	0·7	0·382	84	12	86	Diffused-Air (Brandol 60)
	27	404	462	3600	61	2·1	0·228	138	34	75	INKA-System
	26	418	350	3900	61	1·4	0·304	99	23	77	INKA-System
	24	456	537	4200	51	2·1	0·210	141	38	73	INKA-System
	19	583	412	6000	43	1·1	0·376	84	21	75	Kessener-Br.
	18	600	644	4200	53	2·7	0·208	141	34	75	Kessener-Br.
	16	687	812	5100	42	2·5	0·191	141	41	71	Kessener-Br.

T, aeration time (sewage only), min.
R_h, hydraulic load, gal per ft³ per day.
R_b, B.O.D.-load, lb B.O.D. applied per 1000 ft³ per day.
S.S., suspended solids in mixed liquor, ppm.
Index, sludge volume index (Mohlmann).
R_b/R_{TS}, sludge load, lb B.O.D. applied per lb sludge per day.
R_N/Rb', energy consumption, kWh per lb B.O.D., removed (without return sludge).
B.O.D., infl. from settled sewage, ppm.
* Energy consumption including return sludge pumping.
† Energy consumption too high through experiments.

Wuhrmann (1954) at Zurich and v.d. Emde (1957), showed similar results. Even with an aeration period of 1 hr, the B.O.D. reduction obtained in the experiments at Detmold was still more than 85 per cent. Kuisel (1941) conducted experiments in Zurich and obtained B.O.D. reduction of 80 per cent with a detention period of 1 hr.

With greater hydraulic load and shorter aeration periods, the B.O.D. reduction gradually decreases. B.O.D. reductions of more than 80 per cent were obtained at Kassel and Detmold with a detention period of 45 min. With aeration times of from 20 to 30 min, the B.O.D. reduction was more than 70 per cent. The Kassel observation confirmed Pasveer's (1955) experiments which showed considerable B.O.D. removal after from 10 to 20 min aeration. Eckenfelder (1956) and Katz and Rohlich (1956) have reported on the initial reaction phase. They found that the initial reaction phase is completed after from 10 to 20 min aeration and that a B.O.D. reduction of 75 per cent or more is attained with domestic sewage. Considering a return sludge ratio of 50 per cent in the Kassel experiments, with sewage distribution throughout the whole length of the tank, the actual contact time was from 10 to 20 min. It may be noted that at Kassel and Detmold, there were no arrangements for reaeration of the return sludge.

In continuous operation a B.O.D. reduction of 53 per cent was obtained with an aeration time of about 40 min without considering return sludge (Torpey, 1958). With detention periods ranging from 80 to 180 min, the B.O.D. reduction was 60 per cent. Unlike the experiments described above, the sewage of the Owl's Head plant had not been mechanically preclarified prior to biological treatment. There was also a substantial difference in the sludge content and in the return sludge ratio. Design and construction of the new sewage disposal plant Hamburg-Kohlbrandhoft, which will serve 1·2 million inhabitants, are based on the results of the experiments undertaken by the Department of Sanitary Engineering of the College of Technology (Technische Hochschule), Hanover, and the Department of Public Works in Hamburg. Primary settling will have a detention period of about 8 min to remove heavy solids. In preliminary operation, treating 16 mgd, a B.O.D. reduction of 70 per cent was obtained with an aeration period of 30 min.

VOL. 5 1961

EFFECT OF B.O.D. LOAD

The effect of the B.O.D. load on the B.O.D. reduction at the experiments in Manchester, Kassel and Detmold is shown in Fig. 2. Considerably higher loads on the aeration tanks than in present practice (Haseltine, 1956) are possible. It will be noted, however, that with higher loads the B.O.D. reduction varies considerably depending on the concentration of the sewage. The percentage rate of B.O.D. reduction is therefore affected more by the aeration period than by the B.O.D. load (Fig. 1). If, however, the B.O.D. of the final effluent is plotted as a function of the B.O.D. load, the results show a more favorable correlation (Fig. 3).

A critical B.O.D. load above which the B.O.D. reduction suddenly shows an abrupt decline, was not observed in the experiments carried out in Kassel and Det-

mold. Very high loads may be used where partial treatment will suffice. This also applies to concentrated industrial wastes as shown by Dryden *et al.* (1956). Trial operation in Hamburg with a B.O.D. load of over 1000 lb per 1000 ft³ per day pro-

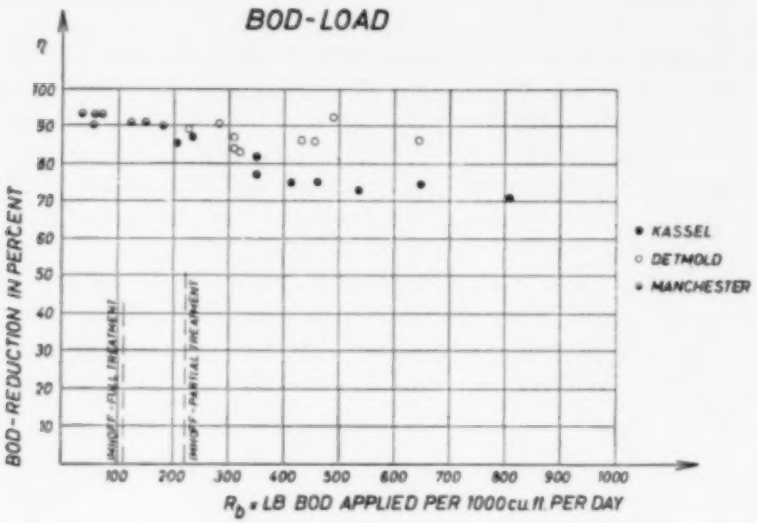


FIG. 2.

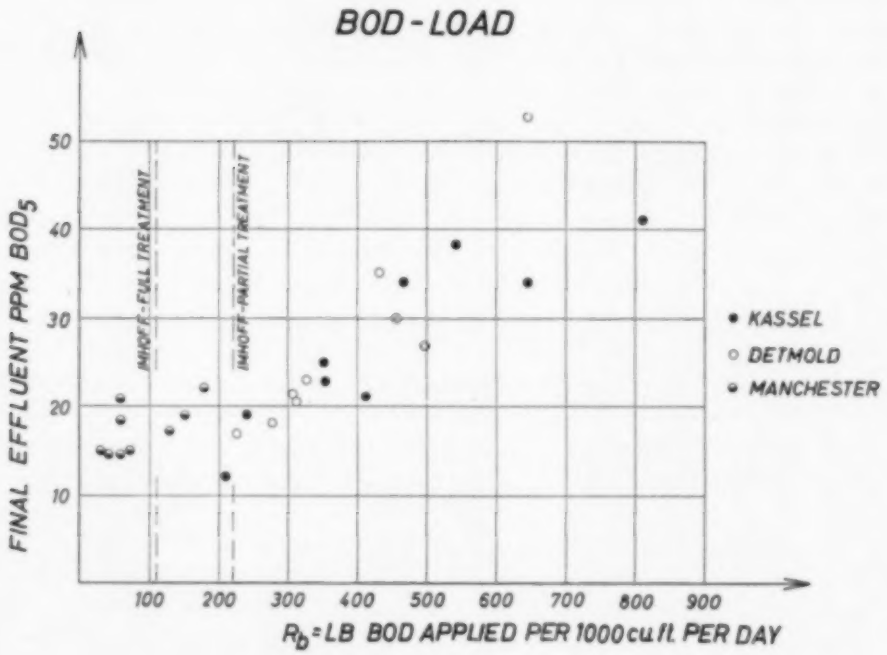


FIG. 3.

duced a B.O.D. reduction of 70 per cent. For B.O.D. reduction of 90 per cent, Imhoff (1958) recommends a B.O.D. load of 110 lb per 1000 ft³ per day and for high-rate activated sludge plants, a load of 220 lb per 1000 ft³ per day. These loads include factors of safety as the experiments at Manchester, Klockenburg, Zurich, Kassel and Detmold have shown. Even with a load of 220 lb per 1000 ft³ per day, it should be possible to obtain a B.O.D. reduction of over 80 per cent and a final effluent B.O.D. of from 20 to 30 ppm.

EFFECT OF SLUDGE LOAD

Purification has been related to the B.O.D. sludge loading (lb B.O.D. per lb sludge per day) (Haseltine, 1956). Fig. 4 shows the B.O.D. reduction related to

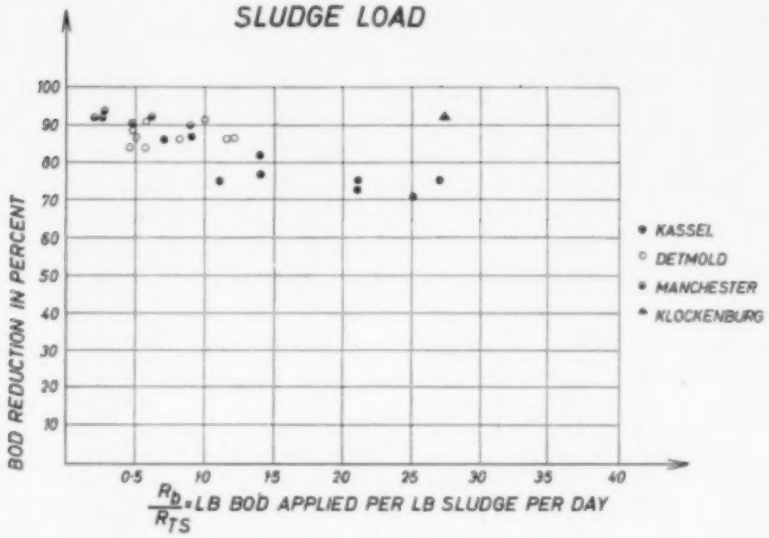


FIG. 4.

sludge load at Manchester, Klockenburg, Kassel and Detmold. There is no appreciable effect up to a sludge load of 3·0. In each case the effect of an aeration period is dominant. At Klockenburg, Pasveer (1954) obtained B.O.D. reduction of more than 90 per cent with a sludge load of 3·0. It appears from present information that for full treatment a sludge load of from 0·5 to 1·5 should be used, and for partial treatment a load of from 1 to 2·5. If these loadings are maintained with high B.O.D. levels, the sludge content in the aeration tank will be high. At the experiments in Kassel and Hamburg, the sludge content in the aeration tanks varied from 5000 to 6000 ppm (suspended solids) and in Detmold, a sludge content of up to 11,000 ppm was maintained. Operation at Detmold and Hamburg using flat rectangular tanks having a length of 250 ft, a surface load 650 gpd per ft², a detention period 2 hr, and a weir overflow rate of 160,000 gpd per ft², has shown that

VOL. 5 1961

it is possible to attain these mixed liquor solids. The primary difference between modified aeration and European high rate activated sludge plants is the concentration of mixed liquor solids. Whereas American highrate activated sludge plants operate with a lower sludge concentration, European practice counter-balances the higher load by using a correspondingly larger sludge quantity (cf. Wuhrmann, 1954). Without preclarification, activated sludge in the modified aeration plants consists of a high percentage of non-biological suspended solids, and consequently, the actual biological sludge load becomes even larger. This fact may be one of the reasons for the difference in B.O.D. reductions obtained.

NATURE OF ACTIVATED SLUDGE

It will be seen from Table 1 that the sludge volume index (Mohlmann Index) does not rise with increasing loads but rather shows a tendency to decline. At the experiments in Manchester, Kassel and Detmold, the sludge volume index varied between 40 and 80. Bulking of sludge caused by *Sphaerotilus natans* was not observed. It must be emphasized, however, that the sewage at Kassel and Detmold was in a non-septic condition, for according to Viehl (1941), the development of *Sphaerotilus* is favoured by septic sewage. At the trial operation in Hamburg, the sludge index due to inadequate primary treatment, showed a value varying between 30 and 40. At the experiments in Kassel, efficiency did not seem to depend on sludge age (ratio between suspended solids in the aeration tanks and suspended solids of the removed excess sludge per day) between 0·5 and 2 days. Similar results were obtained by Lindner (1957) in his experiments with the two-stage activated sludge process. The production of excess sludge in the experiments at Kassel averaged 0·8 lb suspended solids per lb B.O.D. load.

The biological characteristics of the activated sludge are chiefly affected by dissolved oxygen and B.O.D. loading. At high B.O.D. loads, flagellates replaced the ciliates even at high dissolved oxygen levels. With a short aeration period of 20 min, the sludge was predominately bacterial fauna. At a low B.O.D. load a decrease in the numbers of stalked ciliates and an increase in the free-swimming forms and flagellates were observed when a lower dissolved oxygen content was maintained in the aeration tanks. With only traces of dissolved oxygen present, bacteria alone were observed in the sludge.

DISSOLVED OXYGEN CONCENTRATION

Favourable conditions no longer exist for certain forms of bacteria when the oxygen content sinks below a minimum value, and purification is adversely affected. Viehl (1940) showed that purification falls off when the oxygen content drops below 1·5 ppm. This was also confirmed in the experiments undertaken at Kassel and Hamburg. When too low a concentration of dissolved oxygen was present the final effluent contained a larger proportion of suspended solids. The Kassel experiments listed in Table 1 were conducted with an oxygen content of from 3 to 5 ppm. The dissolved oxygen concentration was determined every 4 hr

by oxydimeter (after Toedt, 1955). McNicholas and Tench (1959) found that the quality of the effluent deteriorates if an oxygen concentration of less than 2 ppm is maintained for more than 1 day. Viehl, however, has shown that the purification does not improve in the case of higher oxygen concentrations (e.g. 5·5 ppm)—discounting nitrification. An oxygen concentration of from 2 to 3 ppm should be maintained in the aeration tanks. In many cases, particularly in the case of higher loading, oxygen concentration will be the limiting factor in the degree of purification achieved.

OXYGEN UTILIZATION AND B. O. D. REMOVAL

Oxygen utilization has been related to the B.O.D. removal and to the endogenous respiration of the activated sludge (Eckenfelder, 1956):

$$R'_o = dR'_b + eR_{TS}. \tag{1}$$

The operational results of the pilot plants at Kassel and Gutersloh are shown in

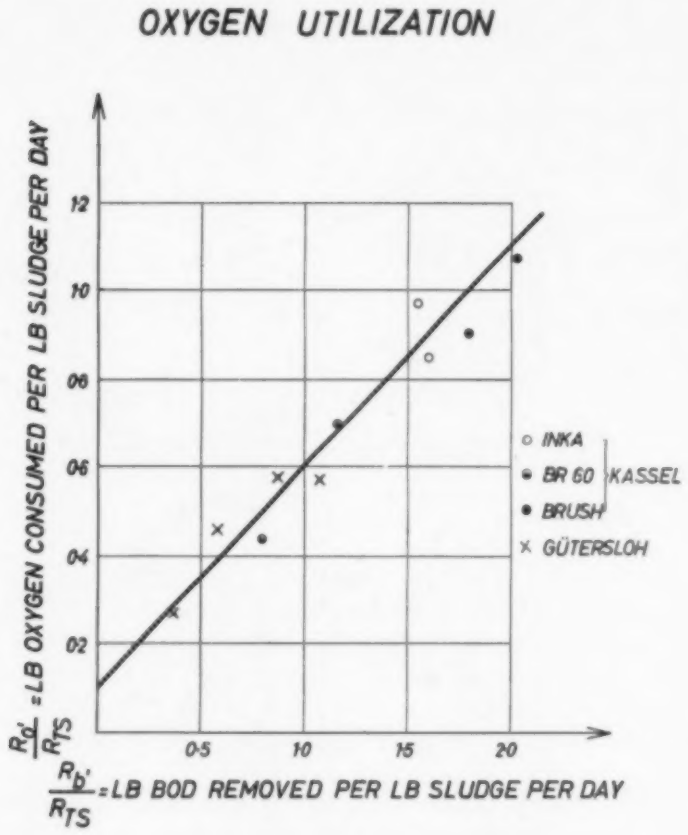


FIG. 5.

VOL. 5 1961

Fig. 5. The relation was as follows:

$$R'_o = 0{\cdot}5(R'_b) + 0{\cdot}1(R_{TS}), \tag{2}$$

and it applies to sludge loadings between 0·5 and 2 lb/B.O.D./day/lb sludge. The studies show little influence of the sludge content R_{TS} on oxygen utilization with higher sludge loads ($R'_b/R_{TS} > 1$). This becomes evident from Fig. 6 where

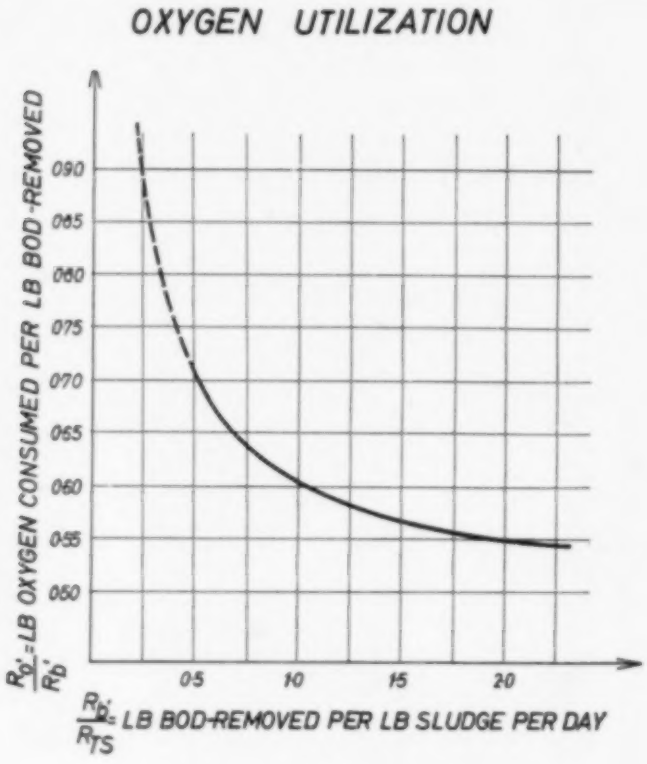


FIG. 6.

oxygen utilization for 1 lb B.O.D. removal (R'_o/R'_b) is plotted against the B.O.D. removal value of the activated sludge (R'_b/R_{TS}). It is therefore most economical to employ high sludge concentrations with high B.O.D. loadings. The above relationship is perfectly in accordance with the relations given by American authors: the coefficients d and e of equation (1) were found to be 0·48, 0·52, 0·53 and 0·07 , 0·09 and 0·15, respectively, by Eckenfelder (1954), Logan and Budd (1956) and Quirk (1959).

The relation given by equation (1) is approximate only, applying to the mean values at the plants where the experiments were carried out. The ratio between the oxygen utilization and B.O.D. removal will be different with each individual

VOL.
5
1961

FIG. 8. Tödt biometer (1955).

type of sewage. Both *d* and *e* will vary in the case of industrial wastes. Oxygen utilization is substantially greater with nitrifying sludges (Ruchhoft *et al.*, 1938).

In Germany, Tödt (1955, 1958) has developed an electrochemical method for measuring dissolved oxygen. The diffusion current which, provided the liquid in the container is intensively mixed (magnetic stirrer), is flowing between the electrode of noble metal (for instance gold amalgam wire) and an electrode of base metal (for instance lead or zinc) provides information on the oxygen concentration in the measuring vessel. The oxygen concentration is measured in micro-amperes. Fig. 7 shows a general section through the instrument, and Fig. 8 a photograph

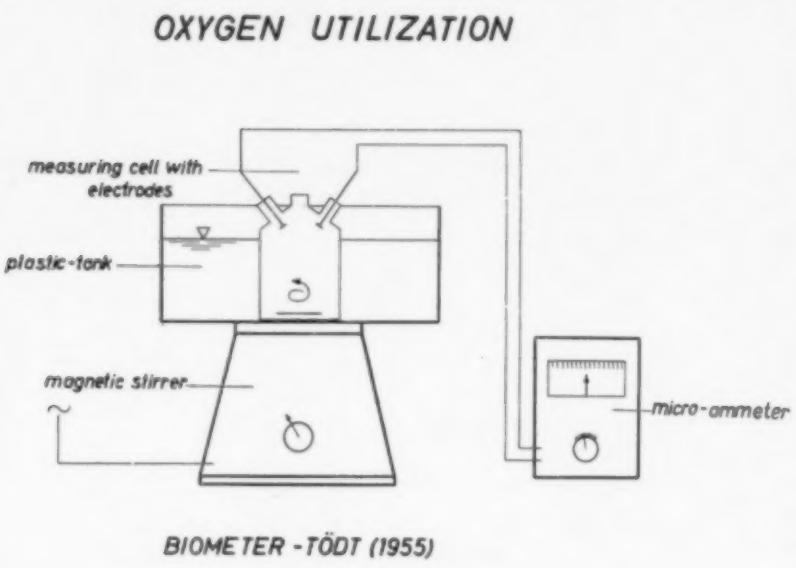


FIG 7. Tödt biometer.

It is portable and easily assembled. The results of some experiments carried out with this so-called "biometer" are discussed below.

DISSOLVED OXYGEN CONCENTRATION AND OXYGEN UTILIZATION

Fig. 9 shows oxygen utilization measurements of an activated sludge fed with dilute milk. In all three cases shown, the oxygen reduction is linear and the oxygen utilization is therefore, independent of the oxygen content. The linear reduction, down to practically zero oxygen content, may be due to the high-speed magnetic stirrer and the consequent intensive mixing.

The excess sludge in the above-mentioned model plant was collected in a second vessel continuously aerated and aerobically mineralized. Fig. 10 shows the oxygen utilization of this sludge, the continuous line representing the oxygen reduction after no excess sludge had been added to the tank for 10 hr. Down to an oxygen content

VOL. 5 1961

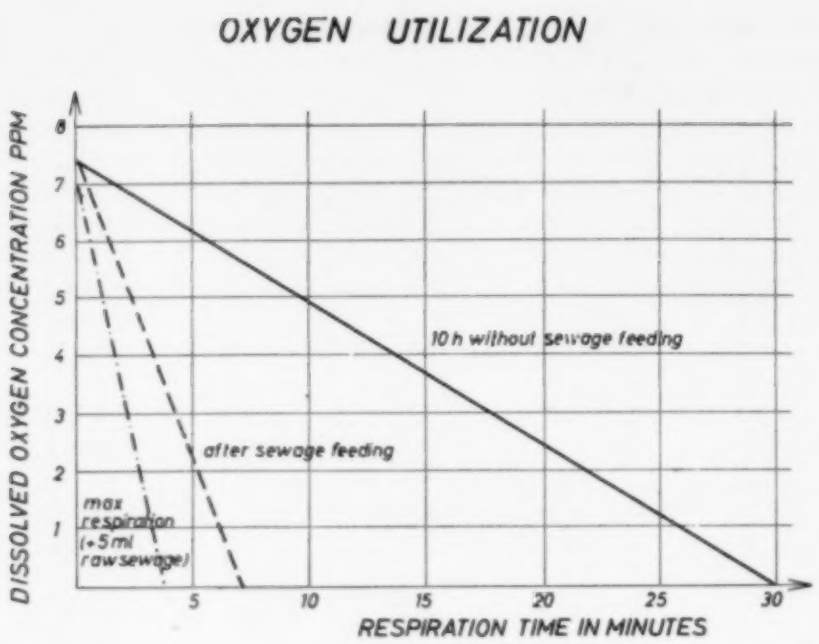

OXYGEN UTILIZATION
DISSOLVED OXYGEN CONCENTRATION PPM
8
7
6
5
4
3
2
1
10 h without sewage feeding
after sewage feeding
max respiration (+5 ml raw sewage)
5
10
15
20
25
30
RESPIRATION TIME IN MINUTES

FIG. 9.

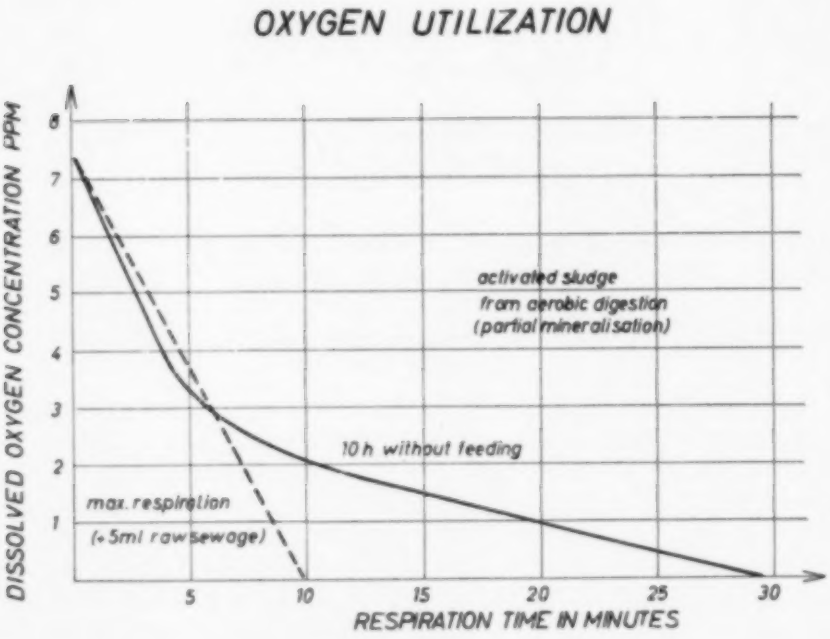

OXYGEN UTILIZATION
DISSOLVED OXYGEN CONCENTRATION PPM
8
7
6
5
4
3
2
1
activated sludge from aerobic digestion (partial mineralisation)
10 h without feeding
max. respiration (+5 ml raw sewage)
5
10
15
20
25
30
RESPIRATION TIME IN MINUTES

FIG. 10.

of 4 ppm, the reduction is linear — below 3 ppm, the rate of reduction deviates considerably from the original. Below 2 ppm, the oxygen reduction is linear again but at a substantially diminished rate. It was observed that the oxygen utilization was independent of the oxygen content when 5 ml of diluted milk was added to this activated sludge. It is known from the literature (Wuhrmann, 1957) that nitrifying organisms place higher demands on the oxygen content, and in the mineralization of the excess sludge, the reactions are predominantly nitrifying. These organisms, however, take no part in the removal of organic matter in the diluted milk. This removal is carried out by the still existing micro-organisms of the excess sludge (the lower part of the respiration curve after aeration for 10 hr without addition of excess sludge). Numerous rotifers were found in the largely mineralized sludge.

ADDITION OF SEWAGE AND OXYGEN UTILIZATION

The sludge of the model plant (Fig. 9) was aerated for 10 hr without addition of diluted milk. The sludge was then transferred into the measuring vessel of the biometer with addition of various quantities of diluted milk. The results of the oxygen depletion experiments are shown in Fig. 11. It is evident from Fig. 11 that

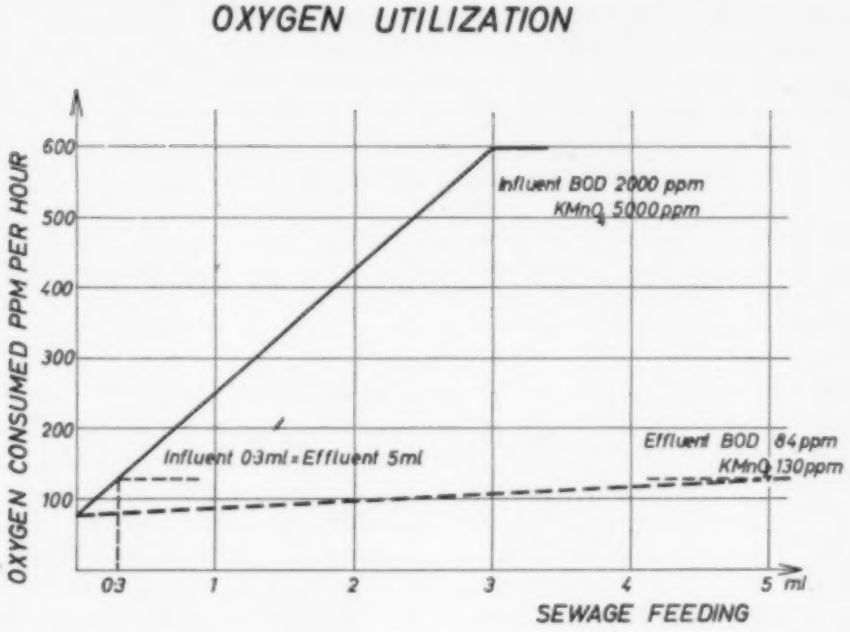


FIG. 11.

the oxygen utilization increases linearly to a maximum and then remains constant with increasing additions of diluted milk (cf. Porges *et al.* 1956). The maximum activity of the activated sludge is reached. Conclusions as to the loading characteristics may be drawn from the ratio between the maximum oxygen utilization value

VOL. 5 1961

and the normal oxygen utilization value. At mean temperature (approx. 15°C), the maximum oxygen utilization value of the micro-organisms in high rate activated sludge plants generally ranges between 60 and 80 mg O_2 per 1000 ppm suspended solids per hr (cf. Sawyer, 1939). If 50 mg O_2 per 1000 mg suspended solids per hr and 1·25 lb O_2 per lb sludge per day are considered reasonable oxygen utilization values, the permissible sludge load would accordingly be about 2·5 lb B.O.D. per lb sludge per day (1 lb applied B.O.D. requires approximately 0·5 lb O_2). This confirms the conclusions arrived at above in the section on effect of sludge load.

Fig. 11 also shows the oxygen utilization values of the activated sludge after additions of various quantities of the final effluent from the model plant (broken line). It becomes apparent from the curve that only small quantities of easily oxidizable substances are left in clarified sewage. Here, too, the oxygen utilization value increases linearly with increasing sewage quantities. In normal activated sludge plants it is easy to determine whether or not biologically oxidizable substances are still left in the final effluent. This type of test can also be used to check the oxidizability of industrial wastes.

TURBULENCE AND OXYGEN UTILIZATION

The effect of turbulence on oxygen utilization has been discussed in detail in the literature (Pasveer, 1954; Gaden, 1956; Porges *et al*, 1956; Wuhrmann, 1959). Oxygen utilization in the biometer was conducted at an intensity of turbulence considerably greater than that found in practice. Varying both turbulence level and mixing time between readings gave very little difference in rate. Stirring for 3 sec per min reduced the utilization rate only about 5 per cent of that with continuous stirring. It would appear therefore that turbulence is not a primary factor in activated sludge plant operation, and that minimum floc diameter is automatically reached with the high oxygen supply per aeration tank unit at high rate activated sludge plants.

ADAPTATION OF ACTIVATED SLUDGE AND OXYGEN UTILIZATION

McNicholas and Tench (1959) believe that a biological adaptation of the activated sludge accounts for the removal efficiency in their experiments at Manchester. An Aero-Accelator with 1·5 hr detention time during the day is in operation at the sewage works at Hamburg-Farmsen. Beside this Aero-Accelator a small experimental tank was being operated with the same type of sewage but with an extremely long detention period of 30 hr. The oxygen utilization value of the activated sludge in the Aero-Accelator during the hours of the day was approximately 100 ppm per hr (about 2500 ppm suspended solids). With an addition of sewage to the measuring vessel, a maximum oxygen utilization value of 230 ppm per hr at 18°C was obtained. On the other hand, the oxygen utilization value of the activated sludge in the experimental tank was about 14 ppm per hr (about 1500 ppm suspended solids) and increased to only 15 ppm per hr after an addition of sewage to the measuring vessel. The activated sludge in the experimental tank is only adapted to the small load and is incapable of accommodating itself to sudden load increases.

In spite of the highly unusual condition in the example under review, it does bring home the fact that special importance must be attached to accommodation of the activated sludge to each individual load. Oxygen utilization measurements to ascertain the efficiency of the high rate activated sludge process should only be carried out with activated sludge adapted to high loads.

OXYGEN SUPPLY

It is necessary to relate oxygen supply to oxygen utilization in order to obtain optimum performance from the activated sludge process. V. d. Emde (1957) and McNicholas (1959) have employed biologically clarified, oxygen-deficient sewage from a final settling tank for oxygenation capacity measurements. This eliminates the need for chemicals to remove dissolved oxygen and excludes any chemicals which might interfere with the oxygen transfer rate (Eckenfelder, 1956). In Fig. 12 the required oxygenation capacity for 1 lb B.O.D. removal is related to the oxygen concentration in the aeration tank for a saturation value of 9 ppm (about 90 per cent

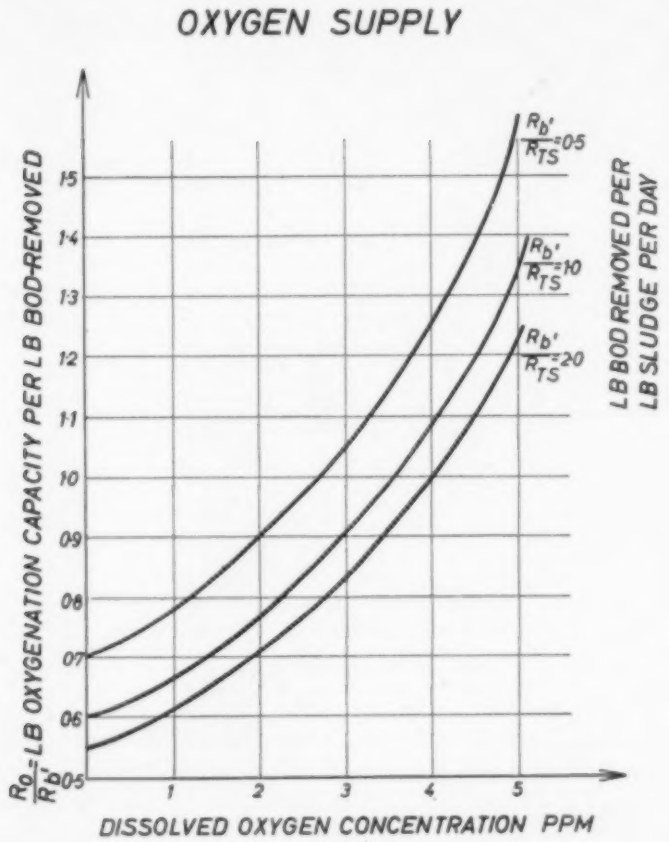


FIG. 12.

VOL. 5 1961

of the saturation value of pure water at 15°C). The oxygenation utilization is based on the approximate relationship in equation (2).

The dissolved oxygen concentration has a greater effect on the required oxygenation capacity than the sludge quantity. A larger oxygenation capacity is required for a lower saturation deficit (for instance with a dissolved oxygen concentration above 3 ppm). For reasons of economy it will therefore be advisable to maintain an oxygen content of from 2 to 3 ppm. A factor of safety of 1·2–1·5 should be provided in the design of aeration devices to cope with peak oxygen demands. A continuous dissolved oxygen recording device (Husmann and Stracke, 1957; Stracke, 1959) is shown in Fig. 13. The revolving electrodes are constantly being ground

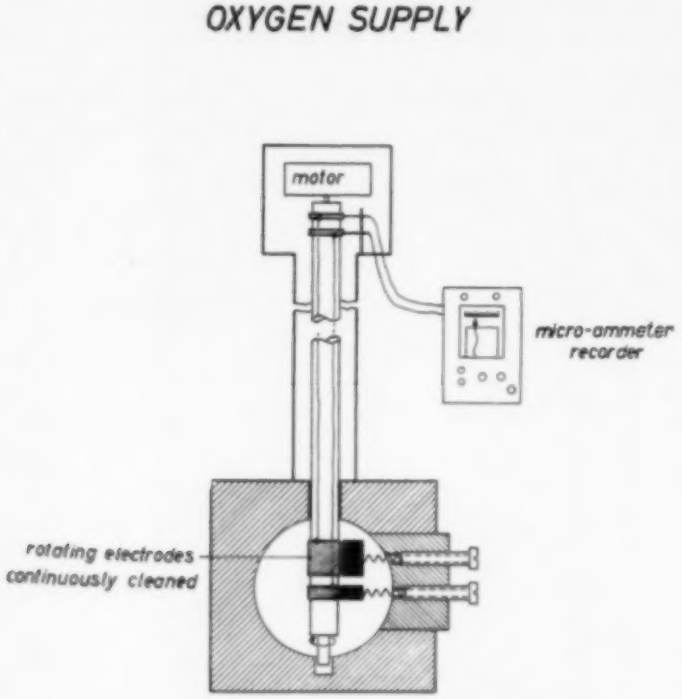


FIG. 13. Stracke oxygen-recorder.

by an abrasive device and thus mechanically cleaned. In Fig. 14, the oxygen recorder is seen in operation on an Aero Accelator at the sewage works at Hamburg–Farmsen. The mixed liquor is pumped from the aeration tank through the oxygen recorder by a self-priming pump. An oxygen recorder working on similar principles has been developed by Stracke for laboratory experiments. A different type, based on

VOL.
5
1961

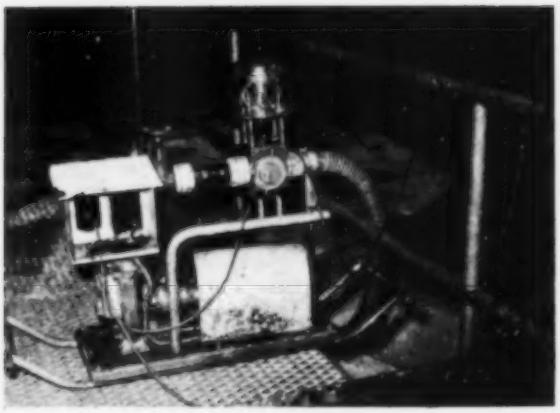


FIG. 14. Stracke oxygen-recorder in operation.

the principle of gas exchange, has been developed by the firm of Chlorator. This recorder measures the dissolved oxygen concentration in the gas phase.

With the aid of continuously working oxygen measuring devices, it is possible to adjust the oxygen supply to the optimum oxygen concentration and so to effect economies in operating costs. In larger plants, the oxygen supply may automatically be adjusted to the oxygen concentration in the aeration tank. In the selection of aeration devices, provision should be made to obtain as readily variable a supply of oxygen as possible.

DISTRIBUTION OF SEWAGE FLOW

The addition of settled sewage distributed along three-quarters of the length of the aeration tanks (according to Schmitz–Lenders, 1952) produced good results at the sewage works in Detmold. An even more favourable effect may probably be achieved by an additional internal circulation (Kessener, 1954) in which part of the activated sludge is returned from the end of the aeration tank back to the beginning to enter the tank with the return sludge. By this means the oxygen utilization is spread equally over the entire length of the tank. A separate reaeration of the return sludge (contact stabilization or biosorption) will probably be necessary in special cases only.

VOL. 5 1961

AERATION SYSTEMS

With equal load and equal oxygen content, approximately the same B.O.D. reduction values were obtained at Kassel with fine-bubble aeration (diffused air-"Brandol" 60 porous pipes from the Schumacher Factory), Kessener brushes (angle-iron brushes and Pasveer's guard attachement of 20 in. diameter as designed and developed by the Passavant Works) and INKA aeration (medium-size porous lattice as designed by Industrikemiska Aktiebolaget, Stockholm, 32 in. submergence) (Fig. 15). For the high B.O.D. load at Kassel, the supply of oxygen had to be high, which meant that the fine-bubble aeration created intensive turbulence in the aeration tank. The floc diameter was correspondingly small. The effect of the additional mechanical agitation on the sludge flocs by the Kessener brush was microscopically detectable. However, the mechanical agitation did not have any adverse effect, although here too, the activated sludge needed a certain amount of time for adaptation. Similar observations were made by McNicholas and Tench (1959) with the Simplex aeration. While the results in Table 1 show that the Kessener brush plant was run at the highest load, and the fine bubble plant at the lowest, it is reasonable to assume that all plants would give equal performance at the same loading providing sufficient dissolved oxygen was maintained. The aeration by INKA lattice, filter pipes "Brandol" 60, and Kessener brushes, produced approximately corresponding amounts of dissolved oxygen for the same energy consumption.

The energy consumption per lb of B.O.D. removed for the three systems corresponds approximately (Table 1) and averages about 0·23 kWh per lb B.O.D. removed

(for settled sewage with a B.O.D. of 140 ppm). The experiments by McNicholas and Tench showed a similar energy consumption for the Simplex aeration. It may also be seen from the table that even with higher B.O.D. loads, the energy consumption per lb B.O.D. removed does not tend to increase. On the contrary, a slight decrease could be observed. All the three aeration systems which are of almost

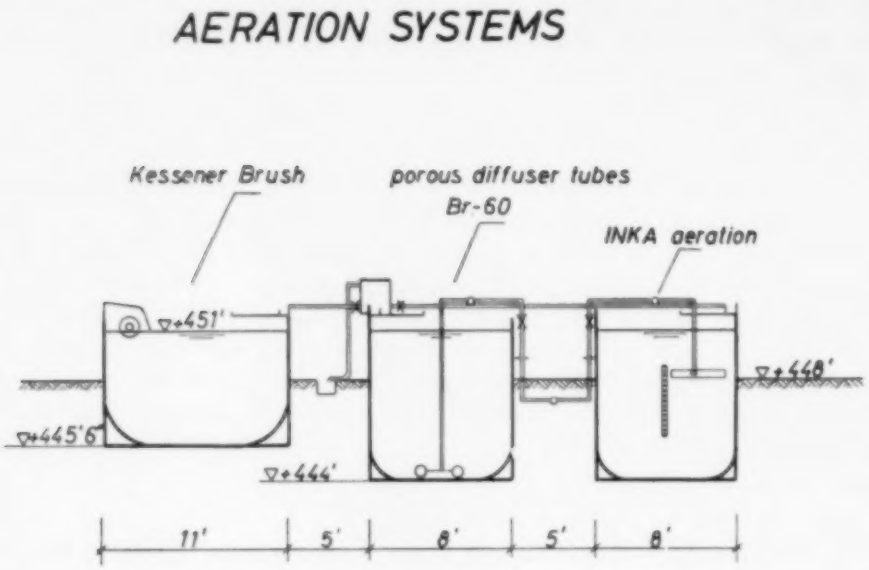


FIG. 15.

equal efficiency have their advantages and their disadvantages from the operational point of view. Porous filter material should not be used for a sewage which favours clogging. The ratio between quantity of air and area of filter should be optimum. With a high oxygen supply, a correspondingly larger number of filters should be installed. With the INKA aeration system, the sewage must be free from fibrous matter in order to provent clogging of the 2·5 mm openings. All head losses should be kept as low as possible by suitably placing the air pipe lines. Only special blowers with a high degree of efficiency should be used. A very large supply of oxygen, e.g. 7 lb per ft length of aeration tank per hr is possible (Fischerstroem, 1958). With the Kessener brush system, the sewage should be devoid of grit. With a high content of detergents in the sewage, special baffles to guide the flow are required in order to prevent dead zones and sludge accumulation in the lower part of the tank. The advantages of this system are that all working parts lie above water and an infinitely variable adjustment of the oxygen supply to the oxygen utilization is possible by merely raising the water level. At places with a high ground water level, the Kessener brush system is particularly suitable as it permits flat construction

of the aeration tanks. A disadvantage is the large number of intermediate bearings and motors in large plants. In spite of this disadvantage, however, the sewage works Hamburg-Koehlbrandhoeft, which is to serve 1·2 million inhabitants, is being equipped with 2000 ft of Kessener brushes (steel plate brushes of the Passavant system). The maximum limit of an economical oxygen supply with the Kessener brush system would appear to be about 3 lb per hr per ft length of tank.

With these three aeration systems, the oxygen supply (continuous operation) appears to range between 3·5 and 5 lb oxygen per gross kWh. Similar results have been reported with the Simplex aeration system as well (McNicholas, 1959). There is thus a sufficient number of economically working aeration systems available with which the operation of a high rate activated sludge plant is possible.

SUMMARY AND CONCLUSIONS

The following observations are made on the major factors involved.

1. *Sewage*

The aeration time has more influence on the % B.O.D. reduction than the B.O.D. load. Partial treatment is possible with very short aeration times (e.g. 30 min). Even full treatment may be obtained with relatively short aeration times (e.g. 60–90 min) and high B.O.D. loads.

VOL. 5 1961

2. *Activated sludge*

Up to a sludge load of about 3 lb B.O.D. applied per lb sludge per day, no direct influence on purification could be observed. In each case, the influence of the aeration time was dominant. With sufficient oxygen content in the aeration tank, the activated sludge will be found to have favorable biological characteristics with many stalked ciliates (*Vorticellae*, *Opercularia*) even in cases of high B.O.D. loads. Even with high loads, a low sludge index was measured.

The great efficiency of the activated sludge is probably due to a biological adaptation of the activated sludge to the high loads (McNicholas and Tench, 1959). It may be advisable that this be taken into consideration when commencing operation of high rate activated sludge plants.

3. *Oxygen*

The oxygen content in the aeration tank should be at least 2 ppm, since below this value, the purification effect will decrease. The oxygen utilization will in the first place be governed by the B.O.D. removal, and to a lesser extent, by the sludge quantity. It is only with lower sludge loads that the sludge quantity will have a greater effect.

Oxygen under normal conditions must be regarded as the *most important limiting factor of the activated sludge process* (Pasveer, 1955). All calculations for oxygen supply equipment should also contain a corresponding safety factor. The following ratios between oxygenation capacity (Kessener and Ribbius, 1934) and B.O.D. load

are therefore recommended:

For full treatment: not less than 1·5 lb O_2 per lb B.O.D. applied.

For partial treatment: not less than 1·2 lb O_2 per lb B.O.D. applied.

4. *Operation*

Distributed addition of sewage along part of the aeration tank with internal sludge circulation may be advisable. The return sludge quantity should amount to between 30 and 100 per cent of the inflowing sewage. The experiments at Kassel with the fine-bubble aeration, Kessener brush, and INKA aeration system have shown almost comparable results. According to the experiments at Manchester (McNicholas and Tench, 1959) the Simplex aeration system has proven to be equally efficient. There is consequently a sufficient number of efficient aeration systems available for high rate activated sludge plants.

Acknowledgements—The author wishes to acknowledge the helpful suggestions given by Professor Dr. Kehr and Baudirektor Dr. Stuewer. Furthermore the valuable help and co-operation of my colleagues on the staff of the Department of Sanitary Engineering of the College of Technology, Hanover and the Department of Public Works, Hamburg is greatly appreciated. My sincere thanks for the translation of this paper into English are due to Herr Schultze and Frl. Drieschner.

REFERENCES

DRYDEN, F. E., BARRETT, P. A. and KISSINGER, J. C., *Biological Treatment of Sewage and Industrial Wastes*, vol. 1, p. 360. Reinhold, New York (1956).

ECKENFELDER, W. W., *Proc. 8th Ind. Waste Conf. Purdue University*, p. 353 (1953).

ECKENFELDER, W. W. and WESTON, R. F., *Biological Treatment of Sewage and Industrial Wastes* vol. 1, p. 18. Reinhold, New York (1956).

ECKENFELDER, W. W., RAYMOND, L. W. and LAURIA, D. T., *Sewage Industr. Wastes* **1337**, 28 (1956).

V. D. EMDE, W., *Veröff. d. Inst. f. Siedlungswasserwirtschaft d. Techn. Hochsch.* Hannover H. 1 (1957).

FISCHERSTROEM, N. C. H., private communication (1958).

GADEN, E. L., *Biological Treatment of Industrial Wastes*, vol. 1, p. 172. Reinhold, New York (1956).

HASELTINE, T. R., *Biological Treatment of Sewage and Industrial Wastes*, vol. 1, p. 257. Reinhold, New York (1956).

HOOVER, S. R. and PORGES, N., *Sewage Industr. Wastes* **306** (1952).

HUSMANN, W. and STRACKE, G., Wasserwirtschaft (1957).

IMHOFF, K., *GWF* 51 (1955).

IMHOFF, K., *Taschenbuch der Stadtentwässerung*, 17. Aufl. Oldenbourg, München (1958).

KATZ, W. J. and ROHLICH, G. A., *Biological Treatment of Sewage and Industrial Wastes*, vol. 1, p. 66. Reinhold, New York (1956).

KEHR, D. and SCHMIDT-BREGAS, F., *Bauamt and Gemeindebau* 240 (1957).

KESSENER, H., *Ges. Ing.* 12 (1954).

KESSENER, H. and RIBBIUS, F. J., *Sewage Wks. J.* **423** (1934).

KESSLER, L. H. and NICHOLS, M. S., *Sewage Wks. J.* 810 (1935).

KUISEL, H., *Interner Bericht der Beratungsstelle an der E.T.H. für Abwasserreinigung*, Zürich (1941).

LINDNER, W., Dissertation T.H. Hanover (1957).

LOGAN, R. P. and BUDD, W. E., *Biological Treatment of Sewage and Industrial Wastes*, vol. 1, p. 271. Reinhold, New York (1959).

McNICHOLAS, J., Rivers Department, City of Manchester, Report (1958–1959).

McNICHOLAS, J. and TENCH, H. B., Inst. Sewage Prif., Conf. Paper No. 1, Bournemouth Conf. (1959).

OKUN, D. A., *Sewage Wks. J.* **763** (1949).
PASVEER, A., *Sewage Industr. Wastes* **1397** (1953); **28**, 149 (1954); **783** (1955).
PORGES, N., JASEWICZ, L. and HOOVER, S. R., *Biological Treatment of Sewage and Industrial Wastes*, vol. 1, p. 35, Reinhold, New York (1956).
QUIRK, T. P., *Sewage Industr. Wastes* **1288** (1959).
RUCHHOFT, C. C., MCNAMEE, P. D. and BUTTERFIELD, C. T., *Sewage Wks. J.* **661** (1938).
SAWYER, C. N., *Sewage Wks. J.* **595** (1939).
SCHMITZ-LENDERS, Ber. Abws. Verein., Wiesbadener Tag. (1952).
STRACKE, G., *GWF* (1959).
TÖDT, F., *Elektrochemische Sauerstoffmessungen*. Walter de Gruyter, Berlin (1958); Angew. Chem. (1955).
TORPEY, W. N. and LANG, M., *J. Sanitary Engin. Div., Proc. Amer. Soc. Civ. Engrs.*, Paper **1681** (1958).
VIEHL, K., *Z. Hyg.* **533** (1940).
VIEHL, K., *Z. Bakt.* II, **161** (1941).
WUHRMANN, K., *Sewage Industr. Wastes* **1**, 212 (1954).
WUHRMANN, K., *Biological Treatment of Sewage and Industrial Wastes*, vol. 1, p. 49. Reinhold, New York (1956).
WUHRMANN, K., *Schweiz. Z. Path. Bakt.* **567** (1957).

VOL.
5
1961

REALISTIC APPLICATION OF THE NEWER DEVELOPMENTS OF AERATION PROCESSES

RICHARD H. GOULD
Greeley and Hansen, Engineers

IN aeration processes, methods have become sufficiently well established to permit realistic applications in major sewage treatment works. For the most part, these newer methods are based on principles and developments originated in the organization responsible for the design, construction and operation of sewage treatment works for New York City, currently, the Department of Public Works. The developments include methods for complete treatment as well as for intermediate degrees of treatment.

BASIC PRINCIPLES

The results of "complete treatment" are expected to be equivalent to those attainable by the conventional activated sludge process. The newer modifications employ the same biological processes, but involve the reaeration of sludge in one form or another. As outlined by the author, in his "sludge age" theory, the success of any "complete" activated sludge method is dependent on the presence in the system of a mass of active suspended solids of from three to four times the weight of the incoming load to be treated but the time of contact of the sewage and the active solids may be as low as 30 min. These principles were first established in his "step aeration" method. They have been confirmed in other installations where different combinations of sludge reaeration and sewage contact periods have been used.

The success of the activated sludge process is dependent on the ability of the final settling tanks to remove 99·5 per cent of the solids discharged to them. For this to happen substantially all suspended and soluble organic matter in the sewage must be incorporated in the mass of activated sludge during its passage through the aeration tanks. Aerobic conditions must also be maintained and the sludge must be held long enough in the system to acquire good settling properties. As stated, a sludge age of from 3 to 4 days is required to meet this latter requirement. The performance of final settling tanks is also affected by sludge density currents and, of course, overflow rates. The greater the concentration of solids in the aeration tank effluent, the greater will be the chance of more of these solids passing over the effluent weirs of the final tanks. This consideration often controls the permissable loading of aeration tanks.

"Step aeration" is the most flexible of the various methods. The addition of increments of sewage to the return sludge as it flows through the aeration tank

VOL.
5
1961

equalizes air demands and the feeding of the sludge. But moderate aeration tank capacity is required, as sludge is aerated at heavy concentrations at the influent end, while effluent concentrations are low enough to assure good final tank performance. The ability to vary the distribution of sewage through the step inlets permits quick adjustment to varying conditions of load and sludge density index. Proportions allocated to sludge reaeration and sewage contact are not fixed.

Complete treatment methods are capable of producing effluents with 10 ppm of suspended solids and B.O.D. For such performance it is important that the ratio of the active solids to incoming load be kept within rather narrow limits, where the biological activity of the activated floc and its settling characteristics are both at an optimum. Prolonged retention of the active floc results in a lowered sludge density index and an aerobic destruction with the creation of finely formed material that becomes detached from the floc and passes over the effluent weirs of the final tanks. Similarly, added material that has been subjected to anaerobic destruction will not all be removed by the active floc. Toxic chemicals and low temperatures will also affect results. Knowledge of requirements and limitations have made complete treatment methods reliable and valuable processes.

The first method for intermediate treatment known as "Modified Aeration" was originated in pilot plant work and developed in full scale plant operation in New York City. It has also been used in other locations.

The term "high rate activated sludge" sometimes applied to this method may have resulted in some confusion. "Modified Aeration" and activated sludge methods, as they have been practiced, have much in common, but operate in essentially different biological ranges. In modified aeration, the active sludge grows in the presence of more food material than it can assimilate and it must be removed from the system before it has time to acquire poor settling qualities. With conventional activated sludge, the food supply is strictly rationed and substantially all is quickly assimilated by the active sludge. The sludge is held long enough in the system so that it feeds on itself and, in the process, acquires good settling properties. The range of operation of the two methods may be best explained by the "sludge age" concept.

Sludge age is a rough approximation of the time that incoming solids remain in the system, and is determined by the ratio of the weight of the active solids in the system to that of the incoming load. The initial growth of activated sludge, in the presence of a surplus of food, is rapid and has a high sludge density index of 2·0 or more. It settles readily. As growth proceeds the sludge density index decreases until, within the approximate age of from 0·75 to 2·0 days, settling properties are so poor that much of the solids are not retained in the final tanks. When held for 3 or more days settling properties are restored. There is, therefore an inoperable range for activated sludge. Modified aeration functions before this inoperable age is reached, and conventional activated sludge operates after it is passed. The closer each method approaches this danger zone, the better the results of each will be.

In modified aeration it has been found that the nature of the B.O.D. load is

important. If much of the load is soluble and particularly of a nature to stimulate the grow of filamentous forms of bacteria, sludge age must be depressed to such an extent as to impair treatment efficiency. With a filtrate B.O.D. up to 40 ppm, removals up to 80 per cent may be expected. With a filtrate B.O.D. of 70 ppm removals may drop to 60 per cent. With filtrate B.O.D. substantially in excess of the latter figure, the process may be unsuitable.

The computation for sludge age takes into account both the aeration period and the concentration of the sludge in the aeration tank. Torpey and Lang have shown that aeration time and solids concentration are interchangeable. Substantially equivalent results were secured with a 2·0 hr aeration period and a 0·5 hr aeration period, by increasing the solids concentration by four times in the latter case. The sludge age was the same for both cases.

Another methcd for intermediate treatment developed in New York City is known as "activated aeration". The method was conceived by Abraham Chasick and developed at the Wards Island plant in an effort to find a method requiring less exacting control and with more stability than modified aeration. It involves the split treatment of sewage in which one part is treated by the "complete" activated sludge process. Excess activated sludge from this part is aerated in the presence of the remaining portion of the sewage. The quality of the combined effluents is the average of the high quality "complete" treatment portion, and the "activated aeration" section.

VOL. 5 1961

At the Wards Island plant this method was operated in parallel with the modified aeration method. The results of the two were quite comparable though rather better in the case of "activated aeration". The action when excess sludge from the step aeration batteries was discharged to the activated aeration section was believed to be largely that of adsorbtion and flcculation. There appeared to be little new biological activity as the air use was only that required to keep solids in suspension. Prolonged aeration periods were of some benefit, but removals seemed to be primarily related to the mass of activated sludge added to the system. As an approximation it may be said that each part of activated sludge will remove 1·0 part of suspended solids and about 0·8 part of B.O.D. in the activated aeration section. Excess activated sludge and sludge removed from the sewage in the activated aeration section are collected in the final tanks of the latter section. The combined sludges may be concentrated to a density somewhat greater than that of activated sludge alone but somewhat less than concentrated sludge from the "modified aeration" method.

RECENT APPLICATIONS

With a knowledge of the possibilities and limitations of all these aeration processes, it is possible for the practising engineer to work out an economical and practical solution to meet best local conditions and requirements. Some major installations applying the basic principles described are outlined below.

District of Columbia

The Blue Plains plant serves the District of Columbia and some nearby areas in Maryland. A study of the capacity of the Potomac River to assimilate sewage effluents showed that a removal of 75 per cent of the B.O.D. in Washington sewage would be required under future expected loadings.

The "modified aeration" process was adopted as being adequate and economical for near future conditions. Aeration and final settling tanks designed for a capacity of 290 mgd were constructed and placed in operation during the summer of 1959.

Present construction includes three aeration tanks and ten final settling tanks. The layout is such that another aeration tank and two more settling tanks may be added. Inlet gates may be also added so that the plant can be operated under the step aeration method in the future. At that time it would also be possible to use activated aeration methods, in case something short of complete treatment will serve. The present aeration tanks provide a 2 hr aeration period at 290 mgd flow plus 5 per cent return sludge. The final settling tanks are rectangular and 250 ft long. The sludge sump for the removal and return of sludge is placed near the half-way point so that sludge density currents will be suppressed before reaching the effluent weirs, located near the far end of the tank.

It will thus be seen that the District of Columbia has been provided with the most economical of the aeration processes and with one that should meet present needs of pollution control in the Potomac River. Should future conditions require higher standards, the methods and facilities for a higher degree of sewage treatment have been foreseen in the plant arrangements provided.

Coney Island Plant—New York City

The effluent of the Coney Island Pollution Control Works is discharged through outfalls, a mile and one half long, to the Rockaway Inlet of Jamaica Bay. The Coney Island beaches are closely adjacent and Jamaica Bay itself provides an important recreational area. It has become necessary to increase the capacity of the works from 70 to 110 mgd.

Present treatment is by sedimentation, supplemented by the addition of chemicals and by chlorination during the summer months. Chemical treatment has become progressively more expensive and does not result in as good effluent quality as is desired for these particular waterways. It was therefore decided to provide secondary treatment by aeration methods for use throughout the year.

A study of the Coney Island sewage and the New York City operating records resulted in the conclusion that the "modified aeration" process would work well at this location. The anticipated effluent is greatly superior to the present effluent and appears to be adequate for the conditions. Plans were drawn up to provide "modified aeration" treatment for the present, with arrangements that would permit full treatment by "step aeration" or by "activated aeration" at any future time.

The works under construction include a new aerated grit chamber, aeration tanks for modified aeration with a 1·74 hr displacement period, additional final

settling tanks to provide an overflow rate of about 890 gal per ft² per 24 hr, separate sludge thickening tanks, and improvements to the present digesters so that they will handle the higher loads. Arrangements have been provided, on a very restricted site, so that if full treatment is required in the future, new primary tanks can be built ahead of the aeration tanks and the aeration tanks can be doubled in size to provide a 2·82 hr displacement period under "step aeration" operation. Hydraulics and conduits are such that split treatment using "activated aeration" could also be practiced. Noteworthy in the present construction is the covering of the aeration tanks with a thin shell roof because of new residential construction across the street. The provision of additional pumps, gas engine generators and blower units within existing structures, and the utilization of existing treatment plant structures, have been a challenge to engineering ingenuity.

Bay Park Plant—Nassau County, New York

In 1951 a 27 mgd activated sludge plant utilizing the "step aeration" method was placed in operation at Bay Park with provisions for doubling this capacity. The rapid growth of the County has necessitated enlargements now under construction to provide for 60 mgd, rather than for 54 mgd, as originally contemplated.

This particular location, and the policies of the County, require that nothing but the most complete treatment will be acceptable. "Step aeration" is therefore to be continued in the plant expansion. From experiences with the County operations, and elsewhere, it was concluded that although capacities were to be more than doubled, it would only be necessary to add 50 per cent to the aeration tank capacity. At design flows plus 25 per cent return sludge, a 2·70 hr displacement period will be provided. Some additional hydraulic capacity will be provided for the old aeration tanks and a single new aeration tank will be built.

A new form of final tank will supplement the existing rectangular tank which had the influent and effluent weirs at the same end, with the sludge draw-off at the far end. These worked well, but under high overflow rates and heavy solids concentrations, there was some short circuiting. The tanks will be modified to provide for "around the end flow": down one flight pass to the sludge sump and return through another pass to the effluent weirs. The new final tanks will be the rectangular straight flow-through type, with the sludge sump just beyond the center point. They will be 264 ft long and similar to the type developed in New York's new Bowery Bay Plant and to those used at Washington and Coney Island.

Yonkers Plant—Westchester County, New York

The Yonkers plant of Westchester County is located on the Hudson River at the North city line of New York City. It receives the flows formerly treated in two fine-screening plants of the County and is designed for a capacity of 60 mgd.

The Hudson River at this location is classified by the Interstate Sanitation Commission as "Class A" water. This means, among other things, that 60 per cent of the

VOL.
5
1961

suspended solids must be removed and that chlorination must be practised during the summer months. The Yonkers sewage is of borderline strength where a 60 per cent removal of suspended solids may or may not be possible with plain sedimentation alone.

The location selected for the works, now in operation, permits receiving two-thirds of the flow by gravity. Because of area and foundation conditions, treatment plant sites along the Hudson River are all expensive. The site has been reclaimed from land under water and has a high cost per unit of area. The provision of area for the conventional type of secondary treatment would have been very costly. Fortunately the work of Torpey and Lang, at New York's Owl's Head plant, had shown that effective removals could be secured with modified aeration down to displacement periods of ½ hr.

With this in mind it was possible to provide a site with room for sedimentation tanks and their future extension, with the expectation that this form of treatment would prove to be adequate. Sufficient room was also provided so that aeration tanks of 1 hr displacement period could be built if required. The plant layout is such that the present settling tanks can be used in conjunction with future modified aeration treatment, and the other plant elements can be adjusted thereto. The County was thus saved much expense in site preparation by these new methods, and is in a position to improve the degree of treatment, should this be required.

Northeast Plant—Philadelphia, Pennsylvania

We are retained by Philadelphia to assist in planning needed extensions to the Northeast plant and on other problems relating to the Delaware River and to sewage disposal in general. Studies of needed extensions at the Northeast plant in Philadelphia are not complete, but some comment at this time is pertinent and of interest.

The Northeast sewage treatment works was designed in about 1947 for a capacity of 125 mgd, using modified aeration process. The plant includes four primary settling tanks, four aeration tanks, each with four passes, and sixteen final settling tanks. Primary and final sedimentation are each provided with a displacement period of about 2 hr, while the aeration displacement period is about 3 hr on sewage flow. Sludge is digested and disposed of in lagoons. It was considered that the plant would remove about 75 per cent of the B.O.D. before the effluent was discharged to the Delaware River but actual results have, in general, provided less than a 60 per cent B.O.D. removal.

Sewage comes to the plant through two main trunk sewers, the Delaware Low Level and the Frankfort High Level. Both sewages contain much trade waste, that in the D.L.L. being the greater. A significant feature, however, is that at times very high values of soluble B.O.D. are present. As discussed above, high levels of soluble B.O.D. cause a rapid growth of filamentous bacteria and cause sludge age to be depressed to such an extent as to impair treatment efficiency. This was probably happening at Philadelphia. It was concluded that, while the Phila-

delphia sewage was amenable to aeration treatment, the modified aeration method could not effect desired removals and other aeration methods were investigated.

The split treatment method was chosen and conduits were separated so that the F.H.L. sewage went to one-half of the plant and the D.L.L. sewage to the other Flows between the two halves could be equalized. The two aeration tanks on the F.H.L. side were altered by a few simple connections so that two passes in each aeration tank could be used for sludge reaeration and the other two passes for contact with sewage and the reaerated sludge. The sludge age in these two tanks was usually kept above 3·0 days. B.O.D. removals of about 90 per cent with effluents of about 15 ppm have been obtained under adequate control. Excess activated sludge from the F.H.L. side has been delivered to the two aeration tanks treating the D.L.L. sewage. Here the excess activated sludge has removed about its own weight of suspended solids and B.O.D. from the D.L.L. sewage, as was expected. When operating within acceptable loading limits of the final tanks, the combined effluents have shown removals substantially above the desired 75 per cent B.O.D. reduction.

Trail operations, started in October 1958 after the completion of the necessary alterations, evidenced great stability with high sludge density indices, despite great fluctuations in both the flow and strength of the sewage received. Available methods for sludge removal resulted in a substantial circulating load within the system. Despite some mishaps related to air supply and maintenance, sufficient information has been gained to indicate that these newer aeration methods will permit the desired purification of a very difficult sewage, and that this can be accomplished within the framework of existing structures and equipment. Plant extensions providing additional capacity will undoubtedly correct some deficiencies in present facilities.

VOL. 5 1961

GENERAL CONSIDERATIONS

Aeration methods are available to produce effluents of any desired quality but changes in future requirements of receiving waterways have an important bearing on present-day plant layouts. For example, although modified aeration may be sufficient for present needs, an increased degree of treatment might be foreseen for the future. If step aeration or a similar method is to be provided, then space for primary tanks as well as for about three times the aeration tank capacity should be provided. Thickening tank areas, air compression and related facilities will be about doubled. To permit the economy of activated aeration, the works would be subdivided to permit split treatment. A flexibility of facilities that permit a variation in treatment with the season should be considered.

A careful determination of the objectives of treatment is essential. Available methods must be considered, not only as to their economy and effectiveness, but also as to their relationship to other treatment plant elements. There is no substitute for careful advance planning.

REFERENCES

1. Torpey, W. N., Practical results of step aeration, *Sewage Wks. J.*, **20**, 781 (1948).
2. Gould, R. H., New horizons for activated sludge. *Engng News-Record* **143**, 180 (1949).

3. SETTER, L. R., Modified sewage aeration—I. *Sewage Wks. J.* **15**, 629 (1943).
4. SETTER, L. R. and EDWARDS, G. P., Modified sewage aeration—II. *Sewage Wks. J.* **16**, 278 (1944).
5. GOULD, R. H., Wards Island capacity increased by structural changes. *Sewage Wks. J.* **22**, 997 (1950).
6. GOULD, R. H., Sewage aeration practice in New York City. *Proc. Amer. Soc. Civil. Engng.* **79**, Separate No. 307 (1953).
7. CHASICK, A. H., Activated aeration at Wards Island. *Sewage Wks. J.* **26**, 1059 (1954).
8. TORPEY, W. N. and LANG, M., Effects of aeration period on modified aeration. *Jour. San. Eng. Div., Proc. A.S.C.E.* **84**, No. SA3, Paper 1681 (1958).
9. TORPEY, W. N. and CHASICK, A. H., Principles of activated sludge operation. *Sewage Wks. J.* **27**, 1217 (1953).

BIO-OXIDATION OF PULP MILL WASTES IN SWEDEN

JAN RENNERFELT

Waste Water Research Laboratory of the Swedish Forest Industries, Sweden

THIS paper reports the results of pilot plant studies on sulfite and boardmill wastes in Sweden. Experiments were conducted treating bleaching wastes and evaporation condensates at the Boksholm mill in an aeration volume of 260 gal in combination with a sedimentation tank. A second series of experiments employed a 21,000 gal aeration tank combined with a turbine aeration unit. The wastes were supplemented with ammonium nitrate and sodium phosphate in the ratios: B.O.D./N = 30 and B.O.D./P = 100. The B.O.D. of the wastes varied between 500 and 3000. The pH of the caustic wastes was about 11 and of the condensates pH 1 to 2. This was adjusted to 7·0. The waste temperature varied from 20 to 25°C in the summer and 15°C in the winter. The M.L.S.S. rarely exceeded 1200 mg/l. The results of this study are shown in Figs. 1 and 2.*

VOL. 5 1961

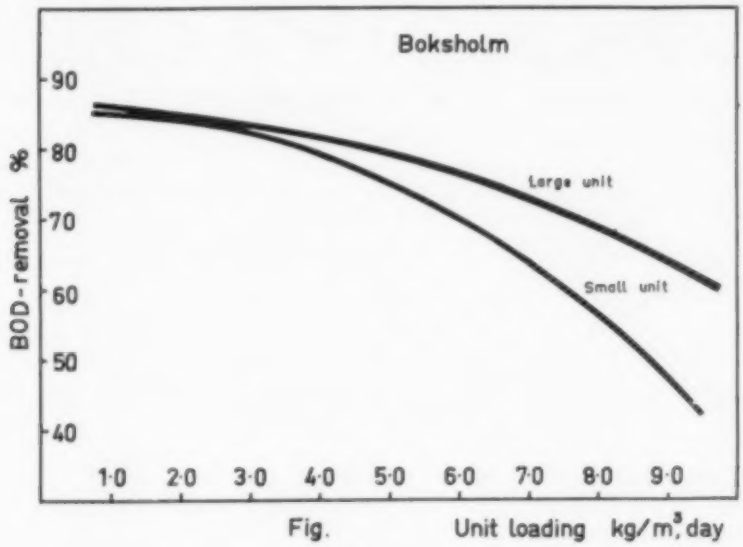


FIG. 1.

Pilot plant studies for boardmill wastes were conducted with an aeration volume of 7500 gal with an associated settling tank. Aeration was conducted with 2 in. drilled pipes. Filtered waste water from the production of hardboard is taken to

* In the larger installation, efficiencies of 85 per cent have been obtained even at loadings exceeding 620 lb/ft³, day.

the aeration tank of the pilot plant. Due to the closed water system in the mill, which is equipped with Asplund defibrators, the B.O.D. of the waste water is as high as 7000 mg/l. Nitrogen and phosphorus is added as 25 per cent ammonia and 8·5 per cent phosphoric acid. The pH is automatically adjusted from 3·9 to 7·0 with NaOH.

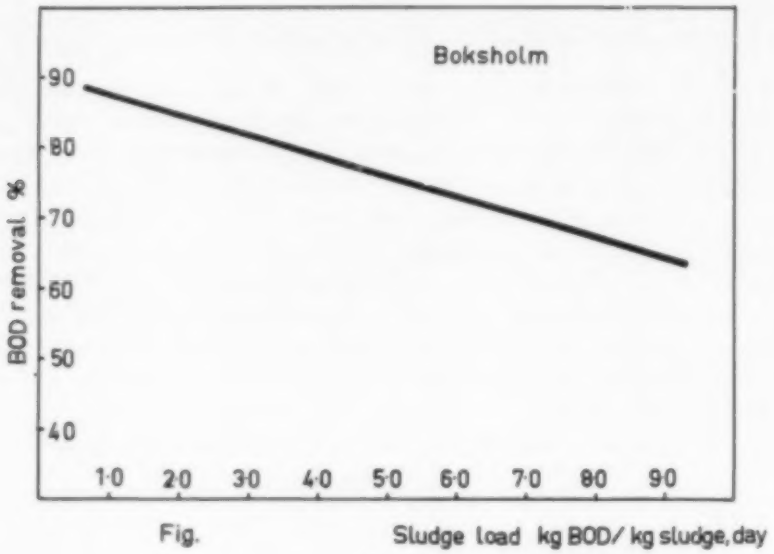


FIG. 2.

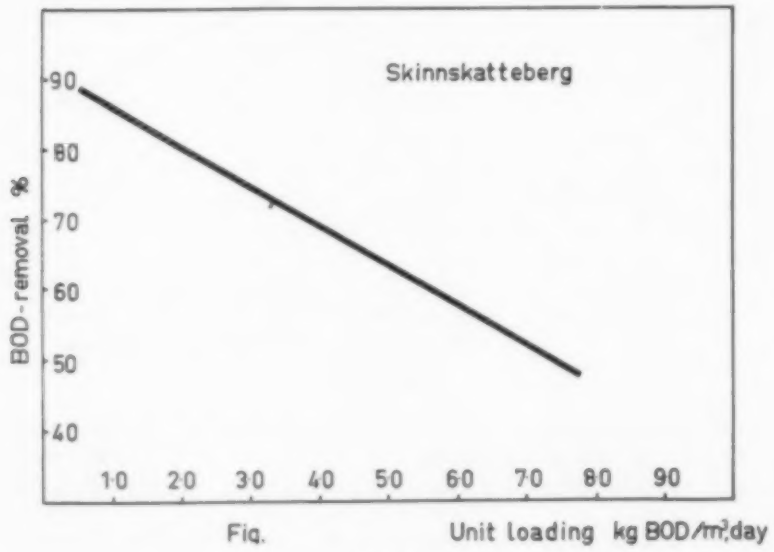


FIG. 3.

A very high return sludge concentration has been obtained, which has permitted a M.L.S.S. of 6000–8000 mg/l. to be maintained in the aerator. The temperatures have varied between 20 and 30°C (68–86°F) during the winter and exceeded 30°C during the summer.

The efficiencies at different unit loadings and sludge loads are shown in Figs. 3 and 4. The unit loading has been varied from 1 to 8 kg per m³ and day (60 to 500

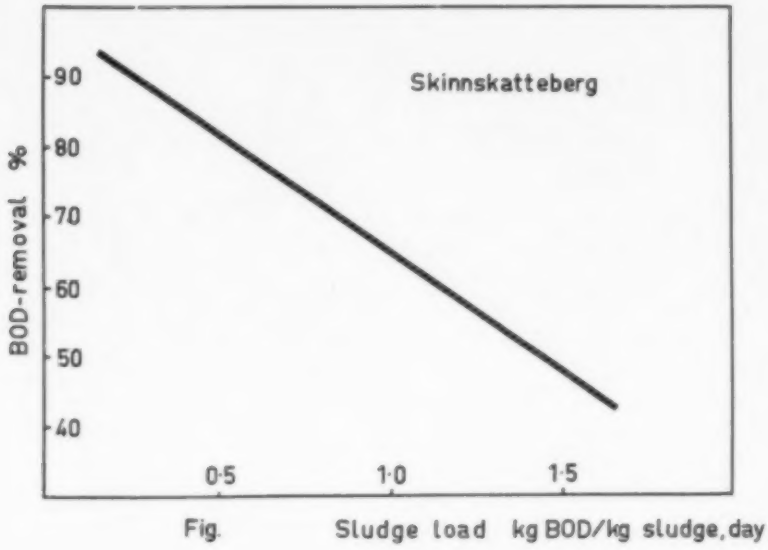


FIG. 4.

lb/1000 ft³/day) and a B.O.D. removal of 80 per cent is obtained at a loading of 2 kg/m³/day. Maximum values of 96 per cent have been recorded. The sludge load in this installation is much lower than in the Boksholm unit. This may be due to the better mixing and aeration in the Boksholm plant.

Experiments with lower N and P concentrations showed that the N and P additions could be decreased from B.O.D./N = 30 and B.O.D./P = 100 to B.O.D./N = 60 and no P with about 10–15 per cent reduction in efficiency. Experiments with no N and P have also been made, but resulted in decreased sludge concentration and low B.O.D. removal efficiencies.

A study of the active micro-organisms is being made at the same time. A normal value of the number of bacteria is 10^9–10^{10} per ml in both the Boksholm and the Skinnskatteberg installations. Even micro-organisms of other types, such as ciliates and fungi, have been found.

We have tried to estimate the treatment costs for biological oxidation of pulp mill wastes. The cost in Swedish Crowns per ton product can be calculated to about 12·00 Sw. Cr. (1962) for a full scale installation at a hard-board mill with a B.O.D. of 60 kg per metric ton board.

VOL. 5 1961

For a Kraft mill with about 40 kg B.O.D. per ton of paper the cost would be about 10·00 Sw.Cr. (1962) per ton (1 U.S. dollar = 5·18 Sw.Cr.). These figures include amortization (10 years), chemicals, power and salaries.

Acknowledgement—The author wants to thank A. B. Statens Skogsindustrier, Skinnskattebergs Bruk and A. B. Klippans Finpappersbruk, Boksholms Sulfitfabrik, who made the construction and operation of the experimental installations possible.

AEROBIC TREATMENT OF TEXTILE MILL WASTES

E. L. JONES, T. A. ALSPAUGH and H. B. STOKES
Cone Mills Corp.

THIS paper reports results on the treatment of textile mill waste in combination with domestic sewage by the contact-stabilization process. The textile wastes are strongly alkaline, highly colored by sulfur, naphthol, indigo, vat and other types of spent dyestuff and contain waxes, gums, oils, dextrines, starches, detergents, bleaching compounds and metallic ions. The suspended solids in the wastes are low.

The average characteristics of the wastes are summarized below:

pH	11·5–11·8
B.O.D., ppm	539–601
Alkalinity, ppm	
Total	639–891
Phenolfthalein	313–515
OH	83–212

VOL. 5 1961

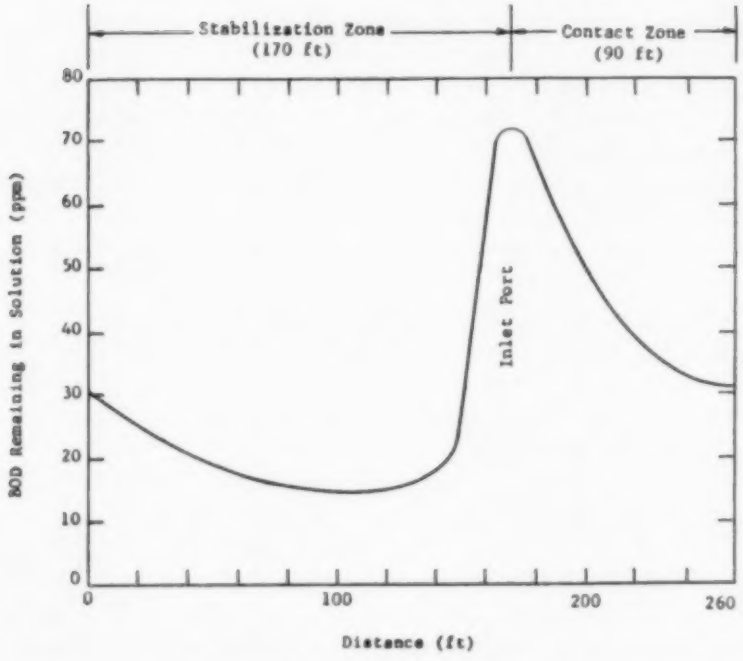


FIG. 1. B.O.D. profile in aeration tank.

No attempt was made to equalize the waste flow from the mills, except for the small periodic dumps of strong caustic and chromium wastes from some of the finishing processes. These were fed into the flow at a uniform rate. Neutralization of the excess alkalinity in the waste was not necessary.

These pilot plant experiments were carried out in an existing 6·5 mgd activated sludge sewage treatment plant using the four primary clarifiers, one of four aeration tanks, and the two final clarifiers. The two zones of aeration (contact and stabilization) were not physically separated.

There was some back-lash of the raw waste as it entered the side of the aeration basin; however, this was not considered to be of significance. The removal of B.O.D. in the stabilization and contact zones is illustrated in Fig. 1 by a typical profile of B.O.D. remaining in solution versus time. This profile was constructed from data obtained during actual operation of the pilot plant.

PLANT PERFORMANCE

A summary of the pilot plant performance data during the time that all the mill waste flow was in the system is shown in Table 1. The data are summarized by operating periods according to the percentage of the total plant influent treated in the pilot plant.

The factors investigated in the pilot plant study included: (1) ability of the biological system to adjust to the mill waste; (2) allowable B.O.D. loading per unit volume of aeration basin for optimum B.O.D. removal; (3) optimum concentrations of suspended solids in the stabilized sludge and mixed liquor; (4) optimum stabilization and contact times in the aeration basin; (5) air required per pound of B.O.D. removed; (6) tendency for frothing in the aeration basin; (7) sludge characteristics; (8) optimum overflow rate in the final clarifier; (9) effect of varying plant flow and B.O.D. load on the color and quality of the final effluent; (10) levels of B.O.D., suspended solids and color removal possible with the process; and (11) effect of primary clarification, pH, caustic alkalinity, and chromium on the performance of the process.

The activated sludge growth developed from the municipal sewage at the start of the pilot plant operation quickly adjusted to change in environment as the mill waste was added into the system. The sludge also quickly recovered from under- or over-aeration and from septic conditions. Sludge bulking, often experienced in conventional activated sludge plants, was not encountered in this operation.

Before all the strong caustic waste was added to the sewer, better than 90 per cent average B.O.D. removals were obtained with average B.O.D. loadings as high as 170## per day per 1000 ft^3 of aeration basin capacity. With all mill waste in the system, removals were 85–90 per cent with loadings as high as 150## per day.

The optimum suspended solids concentration appeared to be in the range of from 2500 to 4000 ppm for the mixed liquor and from 6000 to 8000 ppm for the stabilized sludge.

TABLE 1. CONTACT-STABILIZATION PILOT PLANT DATA (7-DAY AVERAGES) 1958–1959

Treatment period: Month	Dec.	Jan.	Jan.–Feb.	Feb.	Mar.–May
No. of days	31	26	7	24	71
Flow, mgd*	3·34	3·27	2·75	2·67	3·63
Mill waste/sewage ratio†	22/78	19/81	24/76	19/81	22/78
B.O.D. load, lb per day per 1000 ft³ aeration					
Basin capacity	139	127	114	84	122
Return sludge rate, %	70	68	73	84	54
Detention time, hr:					
Stabilization zone	2·9	3·1	3·5	3·5	3·5
Contact zone	1·1	1·1	1·3	0·9	1·1
Final clarifier	3·0	1·6	1·9	1·9	3·2
Air rate, ft³/gal,	1·9	1·8	2·0	2·1	1·8
Air used, ft³/#B.O.D. rem.	752	775	786	1130	916
Clarifier overflow rate, gal/ft²/day	378	742	623	605	412
Settleable solids % of volume	53	35	42	36	30
Sludge index	184	119	121	100	78
Influent					
pH ‡	8·2	8·6	8·4	8·7	8·9
Total B.O.D., ppm	339	320	343	259	277
Dissolved B.O.D., ppm	200	192	203	191	210
Suspended solids, ppm	161	146	164	132	232
Color, ppm	191	329	303	177	196
Total Cr, ppm	1·0	0·6	1·2	0·8	0·8
Phth alkalinity, ppm	38	26	34	50	49
Effluent					
pH	7·3	7·4	7·4	7·5	7·4
Total B.O.D., ppm	36	41	38	39	41
Dissolved B.O.D., ppm	26	27	33	29	28
Suspended solids, ppm	26	24	22	21	27
Color, ppm	54	83	94	63	63
D.O., ppm	0·5	0·8	0·6	0·2	1·1§
Mixed liquor					
Suspended solids, ppm	3081	3326	3511	3810	3974
Return Sludge					
Suspended solids, ppm	7537	7650	7841	8025	12308
Total Cr, ppm	3·4	2·4	3·5	2·9	6·1
% Removal					
Total B.O.D., ppm	89	88	90	86	86
Suspended solids, ppm	85	82	88	84	84
Color, ppm	66	70	68	64	61

* Flows shown were 35, 30, 25, 20 and 30 per cent, respectively of Main Plant influent.

† Ratios do not include week-end flows which contained little or no mill waste flow.

‡ pH not adjusted, week-end values lower averages below daily figures.

§ D.O. increase resulted from sampling station being moved approx. 100 ft downstream from Clarifier Weir to Effluent Weir.

VOL. 5 1961

The stabilization time appeared to be satisfactory over a range of from 2 to 4 hr. The optimum contact time was somewhere between 45 and 75 min. With adequate stabilization time, efficiency of B.O.D. removal was dependent upon the contact time and the mixed liquor solids concentration. Too short a contact time resulted in decreased B.O.D. removal. Too long a contact time improved B.O.D. removal efficiency but had an adverse effect on the settleability of the sludge. The contact time required for the combined mill waste and sewage was almost twice that needed for sewage alone. This was due primarily to the high dissolved B.O.D. content of the mill waste.

The air required per pound of B.O.D. removed averaged from 800–1000 ft^3 which is within the normal range for activated sludge plants using conventional air diffusers. The air blower capacity of the plant and the oxygen transfer efficiency of the air diffuser equipment limited the bio-oxidation capacity of the single aeration tank to approximately 35 per cent of the main plant influent flow or an average flow of 3·5 mgd. Sufficient stabilization and contact time could have been obtained in the existing tank for adequately treating an average flow of at least 4·0 mgd.

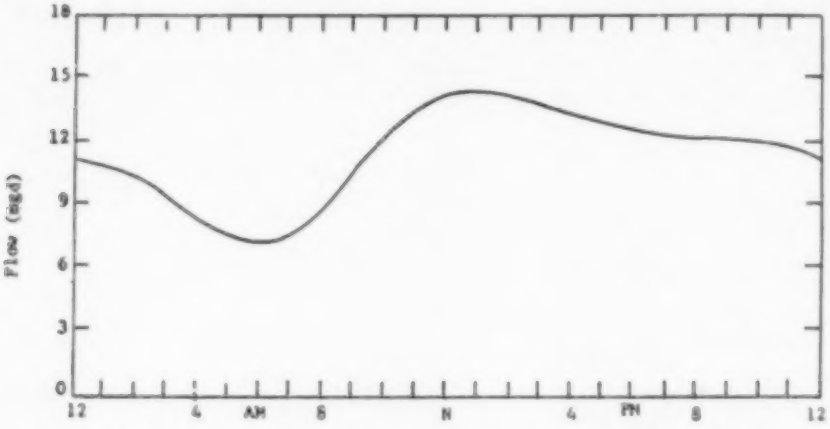


Fig. 2. Hourly variation in plant flow for typical 24-hour period.

Frothing in the aeration basin was not a problem. It occurred primarily in the stabilization zone, increasing during the night as the plant load dropped off and decreasing during the day as the load increased.

With the proper contact time the sludge and the mixed liquor had good clarification and settling characteristics. A sludge index of 120 or less was usually obtained. In a brief trial run in an existing elutriation tank, the excess activated sludge was thickened to a sludge concentration of 2·8–3·2 per cent without difficulty. The supplementary laboratory studies indicated that sludge thickening could be carried to an average concentration of 4·0 per cent.

The maximum overflow rate in the final clarifier was held to about 750 gpd/ft^2. During one period of the pilot plant operation a carry-over of sludge occurred in

the final effluent when the overflow rate exceeded 500 gpd/ft². The sludge was found to have poor settling characteristics and a light flocculent appearance. This condition was corrected by decreasing the contact time in the aeration basin.

To determine the effect of varying plant flow and B.O.D. load, data were compiled over a 4-month period on the B.O.D. of the final effluent at peak daily flow and minimum daily flow for comparison with the B.O.D. of the daily final composite sample. In spite of the widely varying conditions of flow and B.O.D. load the average B.O.D. at peak flow was the same as the average B.O.D. of the daily composites and the average at minimum flow was approximately 8 ppm less. The variations in plant flow and BOD load for a typical 24-hr period are illustrated in Figures 2 and 3.

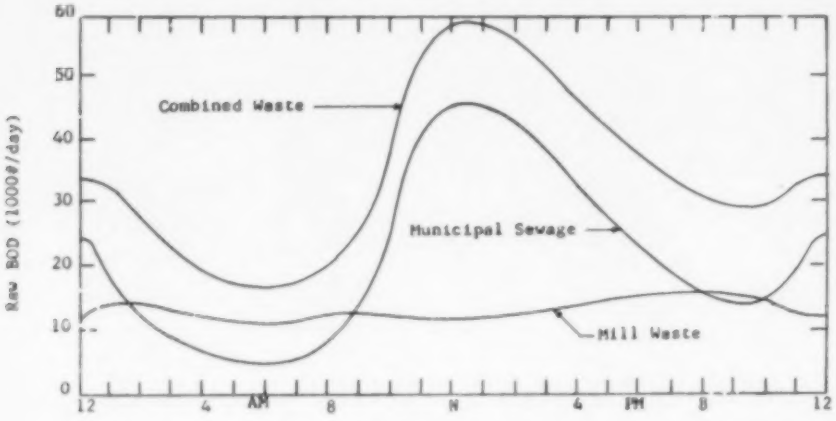


FIG. 3. Hourly variation in plant B.O.D. load for typical 24-hr period.

As will be noted from Table 1, high average levels of B.O.D., suspended solids, and color removal were consistently obtained with the process. Color removal in the waste was exceptionally good. Although the raw waste was highly colored the final effluent had only a slight bluish or yellowish tint. Contrary to what might be expected, the final effluent was clearest at periods of low flows when the ratio of mill waste to sewage was at its highest. The color of the normal activated sludge was bright dark blue. The color changed to gray when the sludge was over-aerated and black when under-aerated.

The pilot plant was operated with and without primary clarification. When the primaries were used some difficulty was experienced in controlling the suspended solids concentration of the stabilized sludge and mixed liquor. To correct this condition, a portion of unsettled raw waste was periodically by-passed directly to the aeration units. After the first few months of operation the entire flow to the aeration basin was by-passed around the primary clarifiers to determine what effect this would have on the system. No difficulty was encountered in controlling the solids concentrations in the aeration unit and the plant continued to operate satisfactorily. Scum was removed at the final clarifiers.

VOL. 5 1961

As can be seen from Table 1 and Fig. 4 the pH of the raw waste was high and fluctuated over a wide range. Apparently because of the low hydroxide alkalinity in the mixed waste and high solids concentration carried in the aeration unit the biological system readily adjusted to this variable.

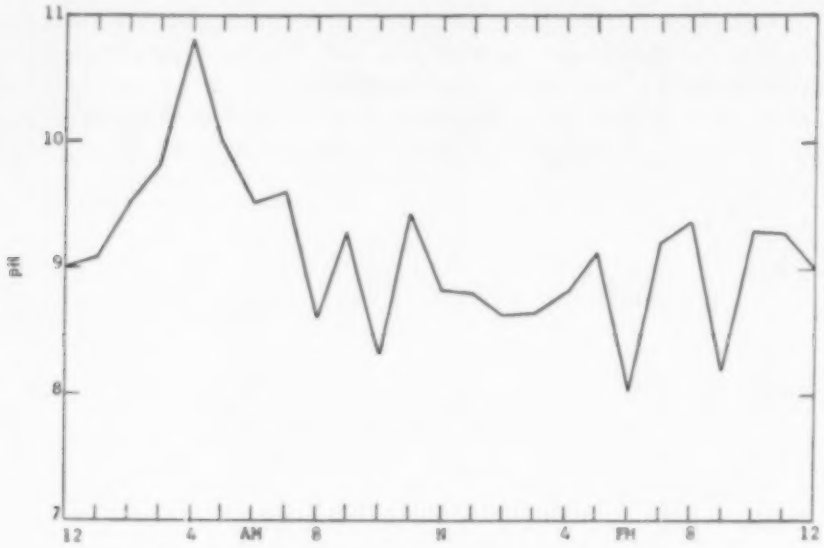


FIG. 4. Hourly variation in pH of raw waste for typical 24-hr period.

The contact-stabilization process, as is true with any activated sludge process, is adversely affected by the presence of caustic alkalinity. The pilot plant, however, performed satisfactorily at the low concentrations encountered in the mixed waste. In the laboratory the activated sludge recovered from shock loadings of up to 150 ppm within a 3 hr stabilization period.

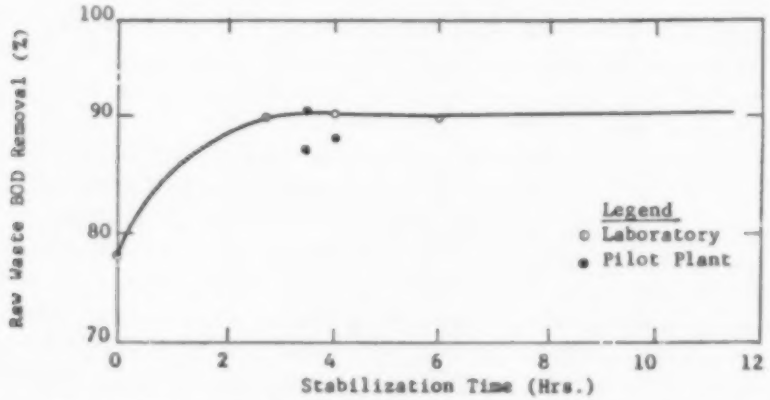


FIG. 5. B.O.D. removal for various stabilization times and 1-hour contact time.

Chromium in the amounts encountered, seemed to have little or no effect on the system. No correlation was apparent in the pilot plant data between efficiency of treatment and the amount of chromium present. The stabilized activated sludge absorbed as high as 10 ppm of tri-valent chromium without noticeable effect.

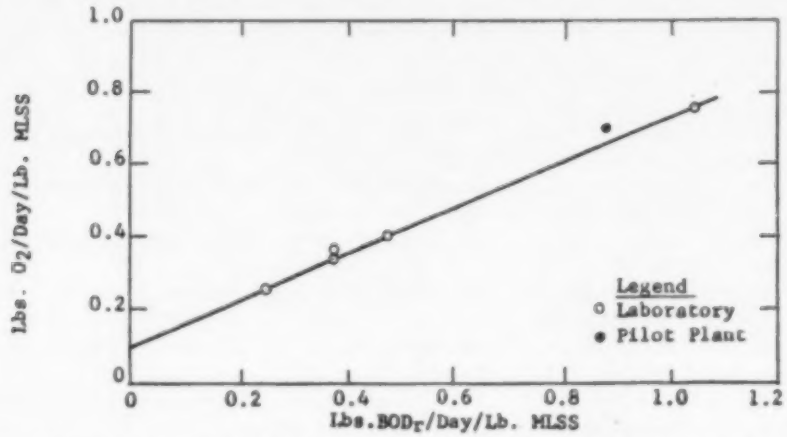


FIG. 6. Relationship between B.O.D. removal and oxygen requirements for various mixed liquor suspended solids levels.

VOL. 5 1961

The composited final effluent sample was chlorinated daily in the laboratory to determine the effect of chlorination on B.O.D. removal. The average of 5 months of laboratory results indicated that from 2 to 3 ppm of chlorine would give approximately 2 per cent higher B.O.D. removals.

The pilot plant data were confirmed by a supplementary laboratory program. Examples of the close correlation between pilot plant and laboratory data for two of the factors investigated are shown in Figs. 5 and 6.

CONCLUSIONS

The performance of the single-stage contact-stabilization activated sludge process in the treatment of combined domestic sewage and cotton processing and finishing wastes compares favorably with that of two-stage bio-oxidation processes.

From the pilot plant studies it was found that with this process: (1) textile mill-sewage waste mixtures having B.O.D. loading ratios as high as 50 per cent mill waste load to 50 per cent domestic sewage load may be treated effectively; (2) B.O.D. removals of 85–90 per cent are possible without chlorination under highly variable conditions of pH, flow, B.O.D. loading, and mill waste concentration; (3) a consistently high degree of color removal is possible; (4) primary clarification of the raw waste is not essential; (5) the bio-oxidation system adjusts readily to high pH, limited concentrations of caustic alkalinity and shock loads; (6) the process variables are easily controlled; and (7) the size of the aeration basins may be substantially reduced from that required for conventional activated sludge.

Acknowledgements—Acknowledgement is made to the following: the Officials, Engineers, and Plant Operating Personnel of the City of Greensboro, North Carolina, for use of the City's treatment plant facilities and for assistance in the operation of the pilot plant; the Management of Cone Mills Corporation for authorization of the work and for encouragement throughout its progress; the Laboratory Personnel of Cone Mills Corporation for assistance in the analytical work; A. H. Ullrich and M. W. Smith, Supt. of Water & Sewage Treatment and Plant Supt. respectively for the City of Austin, Texas, for consultation throughout the pilot plant study.

REFERENCES

1. SOUTHER, R. H. and ALSPAUGH, T. A., Biological treatment of mixtures of highly alkaline textile mill waste and sewage—I. *Amer. Dyestuff Reporter* **44**, 390 (1955).
2. SOUTHER, R. H. and ALSPAUGH, T. A., Biological treatment of mixtures of highly alkaline textile mill waste and sewage—II. *Sewage Industr. Wastes* **28**, No. 2, 166 (1956).
3. SOUTHER, R. H. and ALSPAUGH, T. A., Textile waste recovery & treatment. *Sewage Industr. Wastes* **29**, No. 8, 918 (1957).
4. SOUTHER, R. H. and ALSPAUGH, T. A., Current research on textile waste treatment. *Sewage Industr. Wastes* **30**, No. 8, 992 (1958).
5. ULLRICH, A. H. and SMITH, M. W., Biosorption process of sewage and waste treatment. *Sewage Industr. Wastes* **23**, No. 10, 1248 (1951).
6. ULLRICH, A. H. and SMITH, M. W., Operation experience with activated sludge-biosorption at Austin, Texas. *Sewage Industr. Wastes* **29**, No. 4, 400 (1957).
7. ECKENFELDER, W. W., *Preliminary Report on the Application of the Contact–Stabilization Process to the Treatment of Municipal Sewage and Textile Mill Wastes*. Cone Mills Corporation, Greensboro, North Carolina, (April, 1959). Unpublished.

AEROBIC TREATMENT OF PEA PROCESSING WASTES

R. T. SKRINDE and G. H. DUNSTAN

Washington State University

BIOLOGICAL treatment of food processing wastes has been evaluated by pilot plant studies for many years (1–5). In this study two conventional secondary sewage treatment methods, activated sludge and trickling filters, were evaluated to determine maximum loadings and other information helpful in design of a suitable waste treatment plant. Anaerobic and aerobic lagooning were also investigated on a limited scale.

The waste treated had an average B.O.D. of 574 ppm, a suspended solids content of 135 ppm and a total solids of 1675 ppm. Nearly the entire B.O.D. existed in the soluble state. The chloride content was 544 ppm as Cl . A high variation in B.O.D. existed from day to day.

Activated sludge studies were conducted in a pilot plant consisting of a turbine aeration unit and a settling tank. B.O.D. removal efficiency varied from 90 per cent

VOL.
5
1961

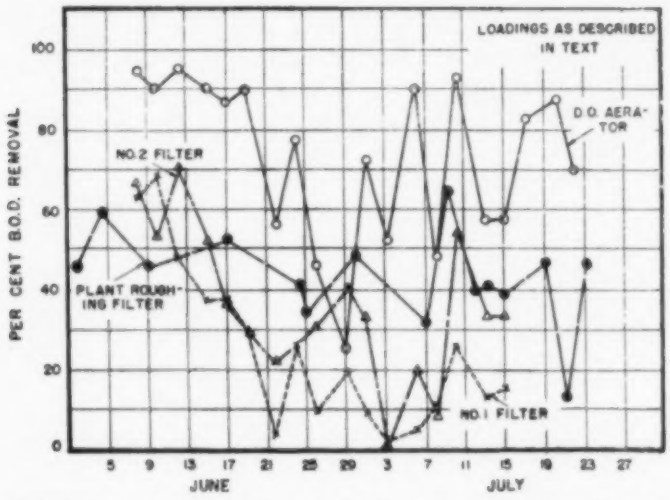


FIG. 1. Efficiencies of B.O.D. removal in pilot units and plant roughing filter.

at the start of operation and decreased to 25 per cent after 3 weeks. This decrease was primarily attributed to sludge bulking and resulting loss in mixed liquid solids. The bulking sludge contained large numbers of *S. natans*. Without bulking nearly 90 per cent B.O.D. removal was obtained at loading levels of 100 lb B.O.D/day/1000

ft³ and 4·3 hr aeration detention. Similar results were obtained employing a cavitator pilot plant.

In the trickling filtration studies, where food processing wastes were treated without sewage addition, low efficiencies were obtained with an organic loading of 418 lb B.O.D./day/1000 ft³ and hydraulic loading of 28 mgad. The filter surfaces were observed to be coated with *S. natans*. Filtration studies were also conducted using varying circulation rates (2 : 1 to 4 : 1) and organic loadings varying from 139 lb/day/1000 ft³ to 194 lb/day/1000 ft³. The hydraulic loading varied from 12·8–19 mgad. At the higher loadings the B.O.D. removal efficiency was less than 20 per cent. At the lower loadings the efficiency approached 50 per cent.

Studies on the sewage plant trickling filter where the cannery waste was mixed with domestic sewage resulted in a B.O.D. removal efficiency of from 45 to 65 per cent. Removals of 246 lb of B.O.D./day/1000 ft³ were obtained. Treatment efficiencies through the various units are shown in Fig. 1.

Treatment resulted in a marked reduction in slime growth in the effluent channel as compared to that resulting from the raw waste.

REFERENCES

1. WEBSTER, R. A., Pilot-plant studies on treatment of vegetable processing wastes. *Sewage Industr. Wastes* **25**, 1432 (1953).
2. DOUGHERTY, R. J., The treatment of food processing wastes. *Proc. 6th Ind. Waste Conf., Purdue University Eng. Bull.* **35**, 479 (1951.
3. ECKENFELDER, W. W. and GRICH, E. R., Cannery waste oxidation by activated sludge. *Biological Treatment of Sewage and Industrial Wastes*, vol. 1, p. 379. Reinhold, New York (1956).
4. O'CONNELL, WM. J., California fruit and vegetable cannery waste disposal practices. *Sewage Industr. Wastes* **29**, 268 (1957).
5. HASELTINE, T. R., Biological filtration of kraut and pickle wastes. *Wat. Sewage Wks.* **99**, 161 (1952).

AEROBIC BIOLOGICAL OXIDATION USING DOWPAC

E. H. BRYAN and D. H. MOELLER

The Dow Chemical Company

PLASTIC media are presently being employed as packing materials for trickling filters. One such material, Dowpac,* a product of The Dow Chemical Company has been under development since 1953. Table 1 lists properties of Dowpac and several conventional and synthetic filter media. (See Fig. 1.)

Dowpac consists of a series of corrugated sheets of thermoplastic bonded together to form a structural packing module. The sheets of Dowpac are shipped nested and then expanded to their packed configuration at the job site. The sheets can be formed in a continuous extrusion–thermal forming operation from any thermoplastic having suitable chemical resistance and structural properties.

The ribs on the sheets of Dowpac space the sheets uniformly 1 in. apart when assembled thus freeing 94 per cent of the packed volume for the build-up of biological slimes and the unrestricted flow of air and liquid waste. The honeycomb formed at the top and bottom of each assembled module of the expanded packing is believed to facilitate uniform distribution throughout the packing so that all surfaces become wetted and support bacterial slimes. The available surface area does not appear to decrease appreciably as slimes build up because nearly all of the packing surfaces are widely spaced and parallel with little possibility for bridging or plugging with excessive slimes. The gradual corrugations prevent free-fall of the liquid waste through the packing but allow the packing to be self-cleaning. Approximately 25 ft of surface area is provided in each cubic foot of packed volume. Dowpac is commercially available made from two thermoplastic materials: high impact polystyrene (Dowpac 10) and Saran copolymer of vinyl and vinylidene chloride (Dowpac 20).†

VOL.
5
1961

PILOT PLANT OBSERVATIONS

Bryan in 1955 presented data obtained in an evaluation of the comparative effectiveness of Dowpac 10 (formerly Dowpac HCS) and conventional trickling filter stone (blast furnace slag).

A tendency for rather prolific slime accumulation seemed to refute the frequently reported need for using media with a rough surface, an advantage frequently claimed for blast furnace slag. The unit was fed only pure phenol which was the only organic carbon source and biologically oxidizable substance present. There was never any evidence of the presence of phenol in the slime or sloughed solids. For phenolic substrates then, the theory regarding biological adsorption, storage and subsequent

* Since publication of this paper the trademark for this media has been changed to Surfpac.

† Now available in U.S. trademarked Surfpac made only from Saran.

TABLE 1. COMPARATIVE PHYSICAL PROPERTIES OF COMMON FILTER MEDIA, EXPERIMENTAL MEDIA AND DOWPAC

Packing	Nominal size (in.)	Units per ft³	Unit wt. (lb/ft³)	Surface area (ft²/ft³)	Void space (%)	Source
Granite	1–3	—	—	23·4	37	(a)(aa)
	—	—	—	19	46	(bb)
	—	—	90·53	29·98	45·3	(cc)
Blast furnace Slag	$2^{9}/_{4} \times 3^{1}/_{2}$ X $2^{1}/_{2}$	51	68	20	49	(b)
Aero-Block Vitrified tile	6″ × 11″ × 12″	2	70	20–22	53	(c)
Straight's Block Theoretical		34·4	56·3	35·8	53·1	(a)
Actual		27·1	44·3	28·2	63·1	
Raschig rings, Ceramic	$1^{1}/_{2} \times 1^{1}/_{2}$ $2^{1}/_{4} \times 2^{1}/_{4}$	340 97	40·8 30·3	35 22·7	68·2 74·9	(a)
Berl saddles, Ceramic	$1^{1}/_{2}$ 2	650 250	42 40	50 32	70 71·5	(d)
Dowpac 10	$21 \times 37^{1}/_{2}$	2	3·6–3·8	25	94	(b)
Dowpac 20	$21^{1}/_{2} \times 38^{1}/_{2}$	2	6	25	94	

(a) Levine, *et al. Sewage Wks. J.*, **8** No. 5, 701 (1936).
(aa) Butcher, *Ind. Chem.* **4**, 446.
(bb) Zeisberg, *Trans. Amer. Inst. Civil Engrs.* Pt. I, **12**, 231.
(cc) Whisler, Unpublished Report, Iowa Eng. Exp. Station, Ames, Iowa.
(b) The Dow Chemical Company.
(c) Aero-Block Bulletin 135, Hugh C. Leibee, Consultant, Minneapolis, Minnesota.
(d) Chemical Engineering Handbook.

oxidation appeared invalid. A materials balance indicated approximately one-half of the phenol fed was fixed into bacterial protoplasm. Cursory bacteriological examination indicated a mixed nondescript bacterial culture predominantly *Pseudomonas* and *Achromobacter* species.

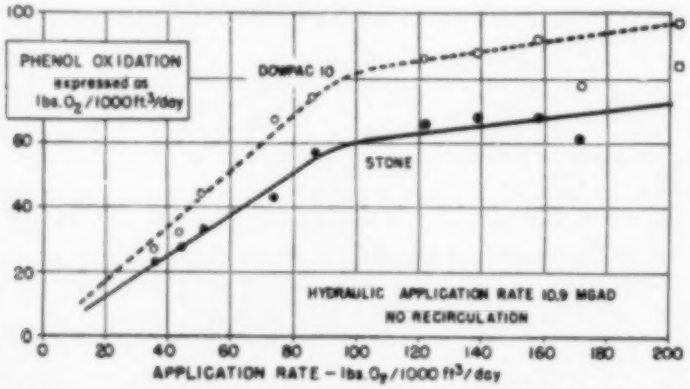


FIG. 2. Comparison of biological oxidation media (no recirculation), pure phenol waste.

VOL.
5
1961

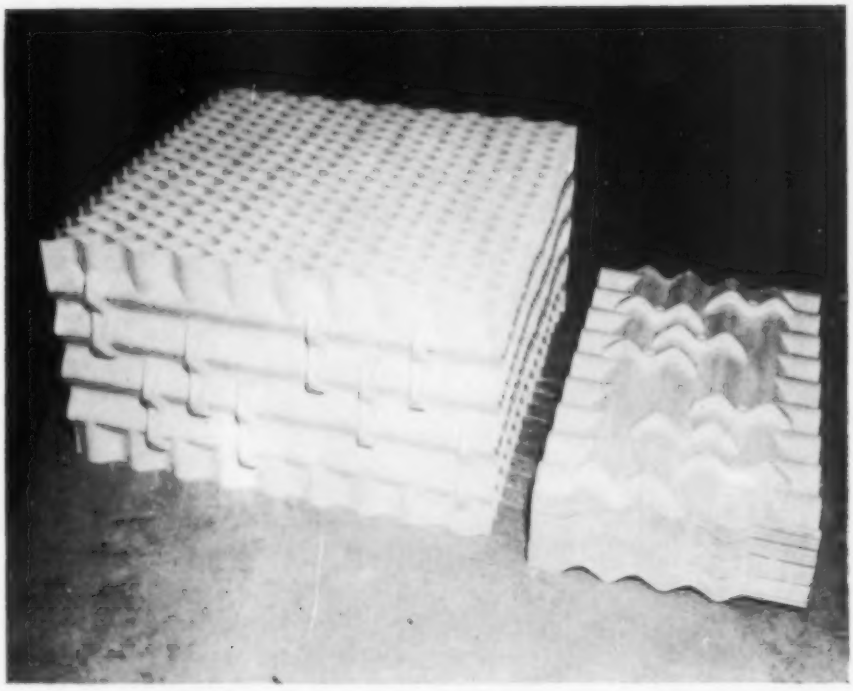


FIG. 1. An assembled, structural module of Dowpac 10 compared with an equal number of nested sheets. The module is in its packing position within the sheets oriented vertically.

Fig. 2 indicates the comparative performance with no recirculation. The slag-packed portion gave a constant phenol removal efficiency of 62 per cent up to a loading of 85 lb of oxygen demand per 1000 ft³ of packing per day. The section packed with Dowpac 10 gave a constant removal efficiency of 82 per cent up to the same organic loading. Influent phenol concentrations at this loading were approximately 180 ppm. The phenol oxidation efficiency dropped off above loadings of 85 lb of oxygen demand per 1000 ft³ per day; however, both the slag-packed section and the section containing Dowpac 10 removed increasing amounts of phenol as the loading increased.

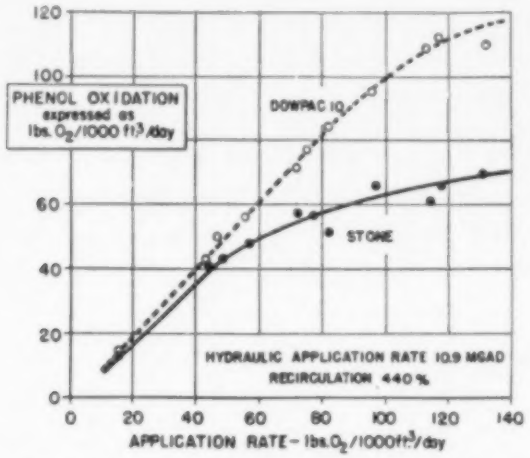


FIG. 3. Comparison of biological oxidation media (with recirculation), pure phenol waste.

Fig. 3 illustrates the comparative effects of recirculation. At all application and recirculation combinations the Dowpac performed consistently better than the slag packed portion.

In numerous pilot and full-scale installations, natural draft aeration has been sufficient to maintain aerobic conditions at all times, even at packed depths as great as 42 ft. Observations of natural draft were made in a 42 ft deep tower in one instance to check on the presence or absence of air flow. Fig. 4 shows the relationship between the volume of air flow versus the temperature differential between influent air and the waste water. Sewage flow to the tower during this study ranged from 0·3 to 2·0 gpm/ft² or from 430 to 2880 gal per day per ft². During the periods of relative stagnation, aerobic conditions apparently still prevailed in the tower.

Short term observations of liquid contact time on unslimed Dowpac have also been made. Hydraulic load cells calibrated to read directly in pounds were placed beneath a 10½ ft deep bottom section in a 21 ft Dowpac 20 pilot tower. After taring the weight of the packing, direct readings of retained waste water plus packing weight were obtained at various hydraulic loading rates. These data are shown in Fig. 5 where calculated contact times in seconds are plotted against hydraulic load-

VOL. 5 1961

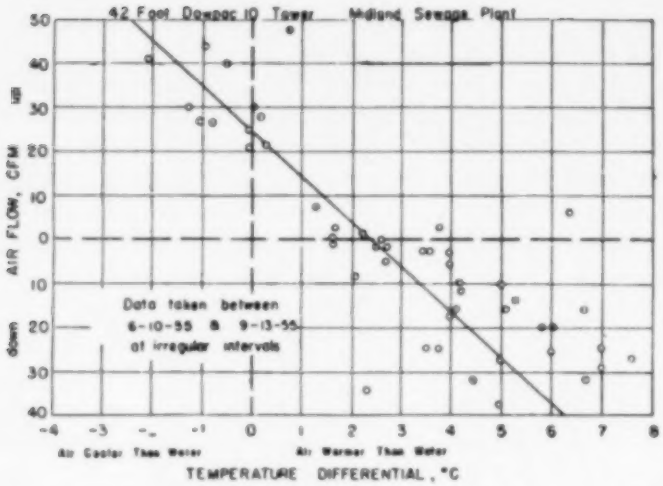


FIG. 4. Variation in air flow with temperature difference. Influent air, avg. water temperature.

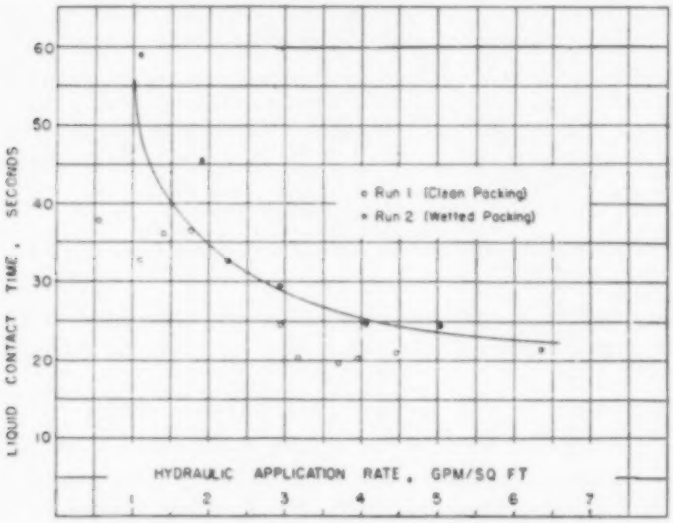


FIG. 5. Liquid contact time, hydraulic rate. Dowpac 20 Tower 10ft–9 in. packed depth.

ings. Run no. 2 was made one day later than run no. 1 which had been made with clean, unslimed packing. It is believed run no. 2 is the more correct since it is likely that complete wetting of the packing was not achieved during run no. 1 because of the initial hydrophobic properties of the clean plastic.

PERFORMANCE

The plastic packing is being applied in a variety of new ways. Typifying deep trickling filter uses, a 31½ ft deep tower treating Kraft paper mill waste gave

B.O.D. removals averaging 75 lb/1000 ft³, day. Hydraulic loading was 77·2 mgad including 100 per cent recycle.

Anderegg (1959) reported on 21 ft deep roughing filter treatment of a phenolic refinery effluent from a sour water stripper. At hydraulic loadings of from 300 to 400 gal/day/ft² (from 15 to 75 per cent recirculation), phenolic removals averaged 75 per cent for 18 months operation. Phenolic concentrations in the feed during this period averaged 228 ppm with extremes of 2 ppm and 2200 ppm. The average of the monthly median of H_2S concentration was 23 ppm with extremes of 0 and 720 ppm. The average daily temperature variation of the mixed feed to the filter unit was 8·5°F with feed temperature variation as great as 52°F occurring in day. The average temperature of the raw feed–recycle mixture was 130°F.

Reynolds and Mauterer (1959) reported on the use of Dowpac as a combination induced draft cooling tower packing and biological oxidation media. The cooling performance was on a par with the conventional wood slat packed sections of the cooling tower. B.O.D. removals in the entire tower (one third of which is packed with 9000 ft³ of Dowpac) are 1800 lb/day from a 6 mgd flow.

REFERENCES

Note: A literature review on trickling filter media was included in the original paper. This review has been omitted in the condensation for this printing but the complete bibliography has been retained as a reference source on filter media.

VOL. 5 1961

Aero-Block Bulletin 135, HUGH C. LEIBEE, Consultant, Minneapolis, Minnesota.

ANDEREGG, F. C., Biological disposal of refinery wastes. *Proc. 14th Purdue Ind. Waste Conf.* (1959).

BLOODGOOD, D. E., TELETZKE, G. H. and POHLAND, F. G., Fundamental hydraulic principles of trickling filters. *Sewage Industr. Wastes* **31** 243–53 (1959).

BUSWELL, A. M. STRICKHOUSER *et al.*, Illinois Water Survey Bulletin 26 (1928). Abstract, *Sewage Wks. J.* **1**, 511–12 (1928).

BRYAN, E. H., Molded polystyrene media for trickling filters. *Proc. Purdue Ind. Waste Conf.* (1955).

CLARK, H. W., Past and present developments in sewage disposal and purification. *Sewage Wks. J.* **2**, 561–571 (1930).

Committee of the Sanitary Engineering Division, *Filtering Materials for Sewage Treatment Plants* ASCE Manual No. 13 (1937).

ESCRITT, L. B., Depth: a possible major factor in percolating filter design. *Surveyor* **111**, 143 (1952).

HOWLAND, W. E., Flow over porous media as in a trickling filter. *Proc. 12th Ind. Wastes Conf., Purdue University* **94**, 435 (1958).

INGRAM, W. T., A new approach to trickling filter design. *Proc. Amer. Soc. Civil. Engr.* Paper No. 999, **82** (1956).

LEVINE, *et al.*, Observations on ceramic filter media and high rates of filtration. *Sewage Wks. J.* **8**, No. 5, 701 (1936),

PHELPS, E. B., *Stream Sanitation.* John Wiley, New York (1944).

REYNOLDS, J. C. and MAUTERER, R. J., Staley saves water, cuts process waste. *Power Engineering* (*February* 1959).

Upper Mississippi River Board of Public Health Engineers and Great Lakes Board of Public Health Engineers, *Standards for Sewage Works* (1952).

VELZ, C. J., A basic law for the performance of biological filters. *Sewage Wks. J.* **20**, 607–17 (1948).

ZOBELL, C. E., The influence of solid surface upon the physiological activities of bacteria in sea-water. *J. Bact.* **33**, 86 (1937).

DISCUSSION

W. E. HOWLAND: Using data of run No. 2 read from the curve of Fig. 5 plotted on log–log paper it has been found that the time varied inversely with the flow to the power n. A graduate student using least square methods found n to be 0·575. Independently another student plotted data that he read from the curve. The curve he located by eye on n was found to be 0·624. If it were thought important to obtain a more precise value of this exponent it would be desirable to apply the least square technique to the original data. In any case it apperas that the value of n is only slightly less than the theoretical value of 2/3.

AUTHOR: Our work with Dowpac has appeared to support the theory that fractional removal of oxidizable organic matter depends a great deal on hydraulic application rate or, if you wish, contact time. Other than the data reported here, we have not accumulated empirical data on contact time to determine if contact time T varies inversely as the 2/3 power of the hydraulic application rate.

We would like to believe along with Professor Howland that a deep filter containing the smallest practical media (or a media like Dowpac which provides very large surface area) is desirable to obtain optimum contact time and maximum efficiency. Some of our data indicate this but we have done no experimental work designed to prove or disprove this as yet.

BIOLOGICAL SLIMES

G. W. Reid and J. R. Assenzo
University of Oklahoma

Biological slimes are very adaptable to environmental conditions. Environment determines the species and the growth rate, and generally the greater the number of species, the healthier the growth. The growth rate will follow the bacterial growth rate curve if started as a new growth, or the unimolecular growth rate curve if prior

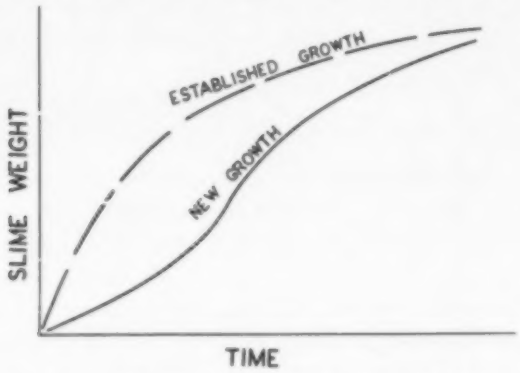


Fig. 1. Growth curves for new and established slimes.

growth has been established. It is this latter condition with which one is concerned in continuously operating waste treatment plants. Fig. 1 shows that, once established, the rate of biological slime growth is quite rapid in the early stages, with a decreasing rate of growth in the later stages. Increased growth occurs if a predicator such as

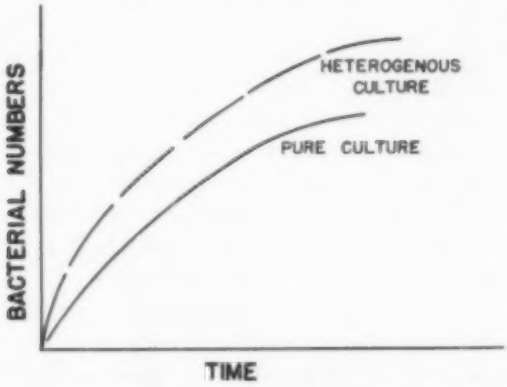


Fig. 2. Growth of heterogeneous and pure slime cultures.

VOL. 5 1961

protozoa keeps the slime vigorous by removing the older material. This can also be accomplished physically by flushing.

In previous studies conducted by the senior author and in the studies made at Oklahoma University, the optimum pH was 7.2 and optimum temperature 27°C. Inclusion of toxic material, or alteration of environmental conditions, tend to eliminate many species and to favor tolerant ones until eventually a pure culture may develop. Such is the case with biological treatment of concentrated trade wastes. *Z. ramigera* are generally considered to be the basic bacterial group in sewage slime. Cells of *S. natans* appear in poorly treated sewage and concentrated phenol will isolate *N. Carolina* as far as growth is concerned. Heterogeneous cultures show a superiority over pure cultures; this is illustrated in Fig. 2. It is a kind of fundamental society where the bacteria consume the soluble food material and the higher forms in turn consume the bacteria, with, of course, the whole chain of interrelated events. A pure culture, specific for one food, does not have these advantages and is difficult to maintain.

ASSIMILATION AND REACTION VELOCITY COEFFICIENTS

Slimes feed by adsorption and then bio-oxidation or assimilation; and surface area and migration distances are therefore important. Experiments in which the pH of the waste was lowered showed that adsorption followed Freundlich type reversibility.

Slime bacteria produce biological oxidation reactions, wherein an elaborate system of enzymes catalyzes the transfer of electrons in an integrated series of chemical reactions from oxidizable substrates to end products. The hydrogen acceptor may be atmospheric oxygen or reducible substances like nitrates and sulphates. Of course, the food to be utilized here must be in a soluble form. The relationship is shown in equation (1):

> soluble food + oxygen or reducible substances + bacteria produce less oxygen or reduced substances + more bacteria and "gum" + carbon dioxide + less food.

The rate of reaction, dy/dt, is usually measured in terms of the slope of the curve of Fig. 1 and formulated in terms of the total growth, L, as: $dy/dt = K(L-Y)$.

K is referred to as the reaction velocity coefficient and, of course, is a very convenient method of evaluating growth. To obtain the growth curve and K value, one may evaluate any specific change in equation (1). For example, one may measure bacterial numbers or slime weight increase, oxygen depletion, carbon dioxide production or nutrient changes. In sewage treatment the oxygen uptake, or alterations in B.O.D. is used. In these studies the uptake of radioactive phosphorous (P^{32}) was used to determine K-values. Biological slimes were grown in a laboratory scale plant which consisted of 2 in. wide by 5 in. diameter glass rotating drums. The slime grew on these drums which rotated in sewage at a rate of about 2 rpm. P^{32} was injected in very small amounts and the phosphorous uptake was traced using a G.M. tube. Fig. 3 shows the laboratory set-up and Fig. 4 is a curve showing the results.

Two situations can be and were studied. The first is similar to an activated sludge plant, wherein the same slime remains with the same sewage throughout the process, or the food supply decreases in concentration with time. The second situation is with the food concentration remaining relatively constant, as would be the case

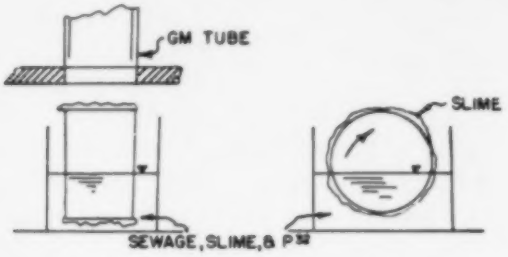


FIG. 3. Apparatus used for growing slime.

of the slime at any particular location in a trickling filter. Slimes grown in this latter fashion were studied for *K*-values, by the P^{32} uptake method, and compared very favorably with actual values determined in the field by oxygen uptake methods (1). Rorschach and Reid (1) obtained *K*-values with synthetic sewage, which agreed with the *K*-values found in similar conditions of sewage treatment.

VOL. 5 1961

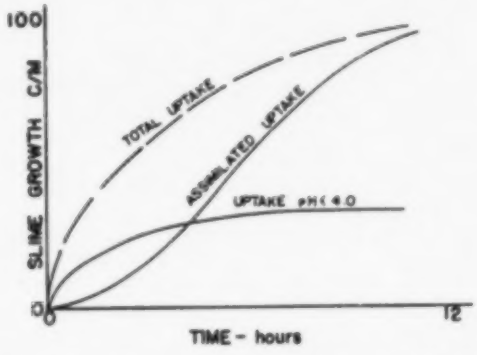


FIG. 4. Slime growth as measured by uptake of P^{32}.

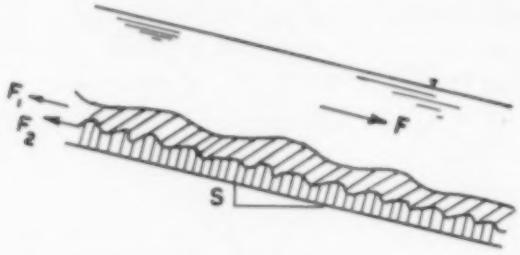
It is realized that P^{32} uptake is a measure of growth while oxygen depletion, the basis upon which *K*-values to date have been determined, is related to respiration. It is felt that there is an interrelationship of the *K*-values determined by both techniques and that they are close enough so that *K*-values determined by the P^{32} technique can be used in place of the *K*-values determined by the oxygen depletion technique. Current research at Oklahoma University is concerned with this hypothesis. It is known, for example, that the *K*-value for oxygen utilization in the B.O.D. test varies widely with temperature, nature of the substrate, and configuration of the apparatus in particular. Also, it is thought that possibly P^{32} uptake may not

be a direct measure of cell synthesis. Consequently, current research is concerned with the development of parallel tests to compare the two techniques and to either establish support for the hypothesis or develop insight into the fundamental differences. The factors being studied and that will be correlated, if possible, are: age of growth, density of growth, food concentration, air–water interface exchange rates temperature, protozoa and other contaminating or predatory fauna, salinity, pH and apparatus configuration.

SLIMES AND ODORS

Slimes will grow on any surface where a water–air interface is created, and where nutrients are available in the water. Consequently, the immersed portion of sewers are covered with slimes and the slimes have a demonstrable relationship to odors produced in a sewerage system.

That slimes attach themselves to "slick" surfaces is due to the development of a primary film between the slime and the "slick" surface. Whether or not the slime continues to grow and develop depends on a secondary film, slime to slime, and whether its cohesive force has the ability to resist the tractive force of the fluid



F = TRACTIVE FORCE
F_1 = ADHESIVE FORCE – PRIMARY & SECONDARY LAYERS OF SLIME
F_2 = ADHESIVE FORCE – SLIME TO PIPE

Fig. 5. Physical and hydraulic forces influencing slime deposition.

above it. This relationship is shown in Fig. 5 and in the following equation for growth in a pipe:

$$\text{tractive force} = RSW = \frac{V^2}{C^2},$$

where R = hydraulic radius,
S = hydraulic energy gradient,
W = unit weight of sewage.

Experiments upon which this was determined were conducted on vertically moving slides, alternately dipping them in sewage or waste, and on sewers, using different flow conditions and different pipe materials. The slime growth tended to be monomolecular. Studies demonstrated that the greater the nutrition, the heavier the growth. These studies indicated that when the growth became quite heavy, the

tractive forces were sufficient to cause the secondary layer to peal or slough. The slough frequency increased with increased food concentration.

The slime in sewage pipes adheres to the pipe surface, and the amount of slime growth is dependent on roughness of the pipe, temperature, strength of sewage, velocity of sewage flow, and time (2). The studies showed that the presence of submerged slime growth is responsible for the generation of odors, including hydrogen sulfide gas and mercaptans. The odors are the gases liberated by anaerobic fermentation of the organic material. In free flowing sewage, the turbulence introduces enough oxygen into the sewage to keep the redox potential above the point at which sulfates break down to sulfides. The rate of production of sulfides is increased with temperature and B.O.D. Sulfides are produced in a pH range of from 6.0 to 9.0; therefore, increases in pH reduce the proportion of hydrogen sulfide produced.

The slime acts as a matrix and the limited amount of oxygen in the flowing sewage is not available to this matrix and is unable to keep conditions in or under the slime layer from becoming anaerobic. The slime layer will build up, then slough and then start over again. The slime growth decreased as the sewer velocity was increased, even when the discharge was increased. The rougher and more porous pipe surfaces sustained a greater amount of slime growth than the smoother surfaces. The velocity was reduced by the resistance offered by slime growth with a consequent reduction in sewer flow capacity.

VOL. 5 1961

CONCENTRATED WASTE STUDIES

A study was made to determine whether or not a specific slime forming bacteria could utilize, on a continual basis, phenol as a food. If this could be accomplished,

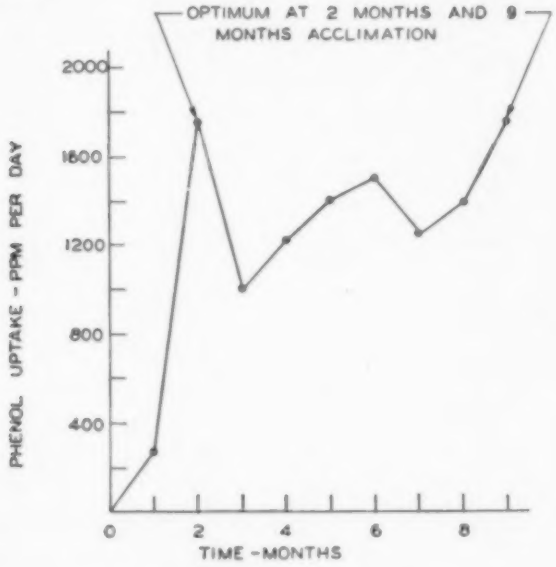


FIG. 6. Drum slime acclimation.

it was envisioned to shift from pure phenol to a phenolic trade waste. Using two types of experimental units, rotating drums and aeration tanks, phenol loadings on established attached and free-floating sewage slimes were gradually increased from 2 ppm to 7500 ppm over a period of 14 months, with a final break-through concentration of 30,000 ppm.

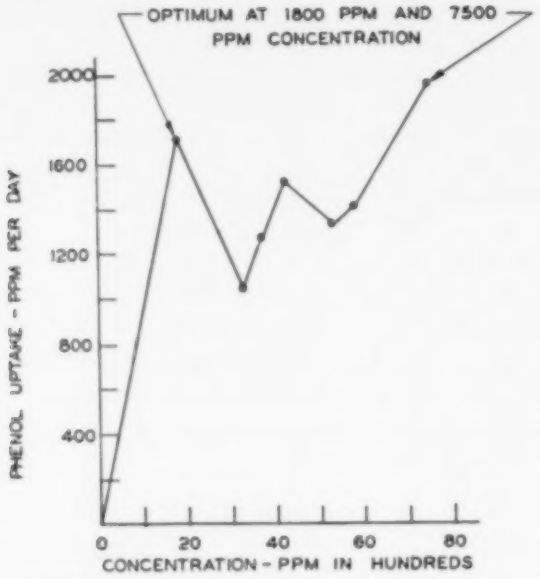


Fig. 7. Drum slime optimum uptake concentration.

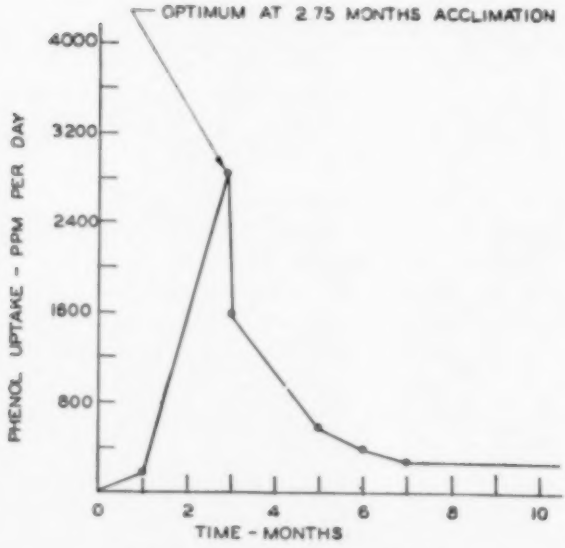


Fig. 8. Aeration slime acclimation.

Fig. 3 shows the laboratory apparatus. Initial studies were operated on a batch basis with the drums and aeration cones using domestic sewage and increasing dosages of phenol followed by continuous dosages. The incremental increases in phenol concentrations seldom exceeded 50 ppm. The end point of the batch treatment was determined by the absence of phenol. The slime was closely observed for physical changes, and reaction coefficients *K*-values, were determined by the use of radiophosphorus. In general, it was found that at least two hours contact was necessary before any

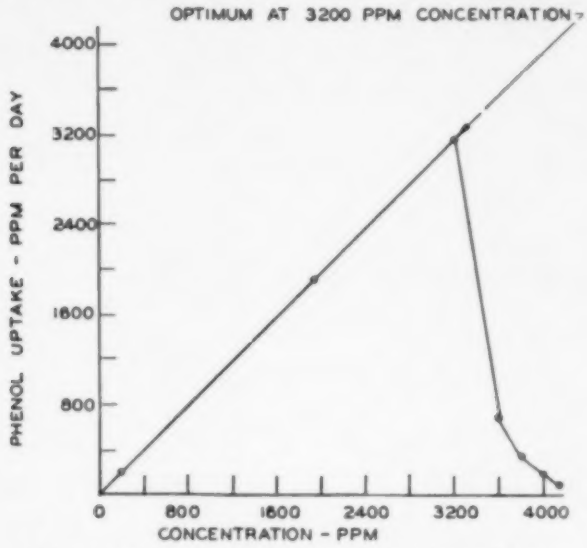


FIG. 9. Aeration slime optimum uptake concentration.

appreciable phenol reduction occurred. Microscopic examination indicated a heterogeneous flora and fauna initially. The analysis indicated a shift to homogeneous growth, color development, and development of poor settling characteristics with increasing concentrations. Studies using *Narcadia corallina* as a pure culture were unsuccessful. The *K*-value, as well as the number of species of bacteria and protozoa, decreased with increasing concentrations.

These studies showed that the drum-grown biological slimes, although they remove phenol more slowly, will tolerate higher concentrations of phenol than will the aeration units. At the heaviest loadings it was necessary to add nutrients (commercial grade 5–10–5 fertilizer) at a concentration of $\frac{1}{10}$ per cent. The slime growth and efficiency of the phenol waste removal rate was not altered by substitution of phenolic trade waste containing oil, cresylic acid, *o*-, *m*-, and *p*-cresols, *o*-dichlorobenzine, and hexavalent chrome. Figs. 6, 7, 8, and 9 show graphically the results of the two units.

CONTAINMENT STUDIES

To determine the effect of chromium included in the trade waste, slimes were treated independently with hexavalent chromium. These experiments were unsuccess-

VOL. 5 1961

ful; consequently, experiments were undertaken adding phenol first, and finally any source of organic material along with the chromium, because of the demonstrated ability of biological slimes to handle industrial wastes containing both chromium and phenol. The chromium was successfully reduced from hexavalent to trivalent and was contained in the slime as long as a source of organic material was included (3).

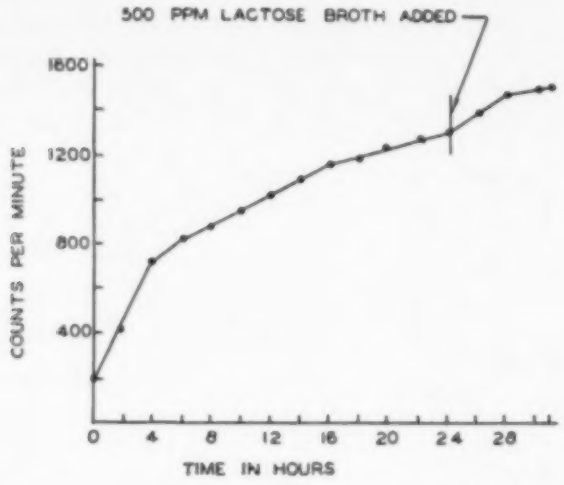


Fig. 10. P^{32} uptake curve for laboratory rotating drum No. 2.

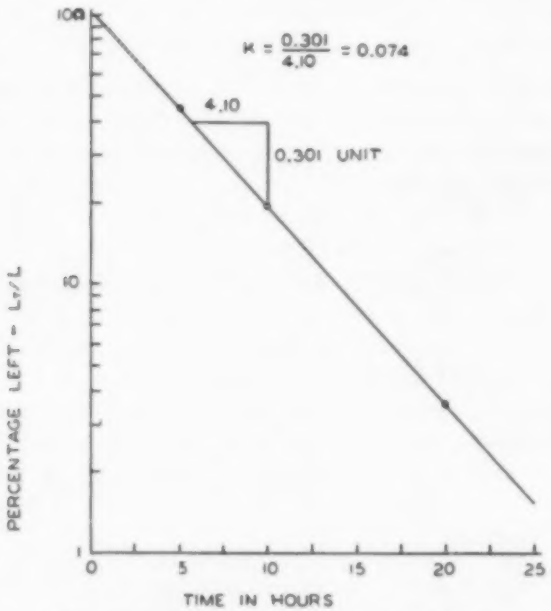


Fig. 11. Percentage oxidation from P^{32} uptake.

As time progressed, under continuous loadings, it took longer and longer for the slime to remove the chromium, until after about 80 days the slimes refused to remove any more chromium. Mass diagrams of loadings indicated that a breakdown of the slime begins at some fixed accumulative chromium concentration. When this occurred the slimes turned black showing growths of mold. Radiochromium, as an index of chromium uptake, and radiophosphorous, as an index of growth, were used together, and a differential β–γ count was taken; (Cr^{51} is a weak γ-emitter and P^{32} a strong β-emitter). The curves obtained were mirror images of each other and the *K*-value was of the same magnitude as previously noted for phenol alone (see Figs. 10 and 11).

SUMMARY

In general, heterogeneous biological slimes display superiority over pure cultures. A change in environment will affect the slime, but the slimes do adapt to new environments, and once adapted, the slime will perform at comparable *K*-values.

The *K*-values determined by the radiophosphorous uptake method compared favorably with published *K*-values determined by the oxygen depletion method.

Biological slimes first adsorb waste products and then assimilate them. It is because of this that treatment processes exposing tremendous surface area are so successful. Removal of waste products is essential; consequently, high volumetric loading rate should be used.

Finally, slimes require oxygen and nutrients and have an optimum pH and temperature range. Because of these, higher loadings are permissible at increased strengths. Also, the addition of organic material will cause the slime in a sewer to grow more rapidly and slough more frequently, thereby reducing odors.

REFERENCES

1. Rorschach, R. and Reid, G. W., A study of biological sewage oxidation using radiophosphorus as a tracer. *Sewage Industr. Wastes* **24**, No. 10 (1952).
2. Reid, G. W. and Yang, T. S., Sewer slimes cause odors. *Wastes Engineering*, September 1959.
3. Wortman, R. L. and Reid, G. W., Removal of hexavalent chromium from liquid trade waste. *S.W. Wat. Wks. J.* In press.

VOL. 5 1961

FUNDAMENTAL FACTORS IN STABILIZATION POND DESIGN

WILLIAM J. OSWALD
University of California

THE scientific study of waste stabilization ponds is now sufficiently advanced to permit formulation of a broadened view of the reactions which occur in such ponds and of their role in the stabilization of organic wastes. It is the purpose of this paper, therefore, to discuss the use of ponds in the stabilization of organic wastes, with special emphasis on certain theoretical relationships which should form the basis for improvements in the design and performance of several types of ponds.

The term "stabilization pond" generally is applied to all bodies of water artificially created or employed with the intention of retaining sewage or organic waste waters until the wastes are rendered stable and unobjectionable through biological decomposition, and the waters are suitable for disposition either by discharge into receiving waters, or by way of seepage and evaporation. Since biological decomposition is the result of the interaction of many processes, a discussion of stabilization ponds should include a detailed consideration of the physical, chemical, and biological processes involved, and of the relationships between the processes. It should evaluate their rates as a function of climatological conditions; and describe methods by which the processes may be fostered and controlled in various pond designs. Before discussing these basic processes, however, it is well to describe briefly the origin and development of ponds, their application, classification, and general performance, and to consider certain physical dimensions and loading parameters which are characteristic of all stabilization ponds.

VOL.
5
1961

HISTORY AND DEVELOPMENT

The purposeful addition of wastes to ponds as a means of disposal was begun in prehistoric times and has continued until the present. During ancient times in the Orient and in Europe, and at present in many places throughout the world, ponds have been built and operated to encourage algal growth by the addition of organic wastes as well as inorganic fertilizers and thereby greatly increase the areal yield of fish that feed directly or indirectly on algae. An interesting presentation of the history, technology and philosophy of such fish culture ponds from ancient to modern times has been given by Edminister (1). During the past 50 years the purification of sewage in fish ponds has been a recognized art in Germany (2, 3). In America, however, such ponds have not been used for sewage treatment.

The first stabilization ponds in the United States evidently were built, not as treatment devices, but for the sole purpose of excluding waste waters from intrusion

into places where they would be objectionable. Once built, however, the waste purification potential of such ponds was increasingly realized. Since Gillespie's (4) description of the ponds of Santa Rosa, California, which were built in 1924, there has been a succession of papers describing one or several specific pond installations (5–8) and a number of papers and articles placing pond design on an increasingly rational basis (9–12). A review of these and many other papers on stabilization ponds has been published by Fitzgerald and Rohlich (13).

It is now generally recognized, that when properly designed, stabilization ponds will become populated with suitable organisms, which will convert waste waters to effluents of a quality equal to those of treatment processes costing several times as much.

APPLICATION

Stabilization ponds should be regarded as a conventional form of sewage disposal and, because of their great cost advantage over other types of treatment, should be the first method to be considered when making a choice among the various available disposal processes.

Stabilization ponds are now employed to treat domestic sewage and many types of agricultural and industrial wastes throughout the United States and in most civilized countries of the world. According to the A.S.C.E. (14) in 1957, Thomas and Jenkins reported the existence of 631 municipal stabilization ponds in the United States, a number which was probably more than doubled between 1957 and 1961. The number of waste ponds in other countries and the number of farm and industrial waste ponds in the United States and elsewhere have not been reported, but it is unquestionably in the thousands.

CLASSIFICATION

Pond types that have been developed include "anaerobic lagoons", in which anaerobic fermentation processes predominate; "facultative ponds", in which anaerobic fermentation, aerobic oxidation, and photosynthetic reduction processes occur at varying rates; and "high rate ponds", in which oxidation and photosynthesis are balanced to yield completely aerobic stabilization, and if desired, a reclaimable excess of algae. The anaerobic type of lagoon has been described by Parker, *et al.* (6); the facultative type by Caldwell (9), Van Heuvelen and Svore (10), and by Hermann and Gloyna (12), and the high rate type by Oswald and Gotaas (11).

Among the pond types listed, that which will be most useful in a given instance depends largely upon land costs and performance requirements. Thus, where ponds are to be located in remote areas, less consideration need be given to odors, and thus, anaerobic ponds may be used. In situations where odor nuisance must be avoided, the use of properly designed and operated facultative or high rate ponds is indicated. However, if the amount of available land is inadequate, or if other conditions preclude the use of ponds, even though properly designed, other types of sewage treatment should be considered.

PERFORMANCE

As with other forms of treatment, inadequate waste treatment in a pond is indicated by the presence of one or more of the following in the pond itself or in its effluent: sight and odor nuisance, unstable suspended organic matter, unstable dissolved organic matter, high B.O.D., excessive numbers of coliform organisms of fecal origin, and fly and mosquito larvae.

Although of little health significance, the complete absence of odor and sight nuisance is most desirable in pond performance because of the need to maintain good public relations. Therefore, a pond that creates an odor nuisance in a populated area for any length of time must be regarded as having failed. Odors from failing ponds are more noticeable than those from other types of poorly functioning treatment plants, because the large surface area of the ponds liberate a proportionately larger quantity of foul gases. Moreover, the presence of hydrogen sulfide and other objectionable gases not only may damage property, but also will lead to overwhelming public pressure to abandon ponding in favor of better understood biomechanical processes.

Unstable suspended organic matter sometimes discharged from ponds consist primarily of accumulated algal cell material mingled with suspended grease or sludge and dead and decomposing invertebrates. Rafts of these materials may form near pond margins as a result of surface skimming by gentle breezes. These rafts usually disintegrate and settle during vigorous wave activity; or, they may be discharged into the receiving stream by way of the overflow. Should the rafts not be destroyed or discharged, however, they may aggregate until a mat is formed that is several acres in extent and several inches in thickness. Conditions favorable to the development of *Clostridium botulinum* may exist in very large and thick rafts. Consequently, there is the danger that wildlife will have access to the toxins formed by the Clostridia, or that the toxins may be discharged into a receiving stream. This danger increases with increasing duration of the condition within the pond. The rafts also may be comprised of or may furnish a site for the growth of blue-green algae, organisms which also may be toxic. Facilities should be available, therefore, for dispersing such materials, or for discharging them into the receiving stream before aggregation and anaerobic decomposition take place. Although performance problems of this type are most frequently associated with overloaded ponds, they also occur in ponds that are properly loaded, but are too large in area, and which grow excessive quantities of algae.

The discharge of live unicellular algae into receiving streams requires consideration. As long as the algae are dispersed and the stream has a velocity of 1 ft per sec or more, the algae will not settle; and if nutrients (i.e. B.O.D) are present, they will continue to grow and to produce oxygen. If clumps of algae are discharged into a lake or into a deep sluggish river, they may settle and with relatively little harm enter the zone of benthal decomposition; or, again, they may form rafts. If the latter conditions exist, and the amount of algae warrants, consideration should be given to the separation and separate disposal of the algae. If the amount of algae

VOL. 5 1961

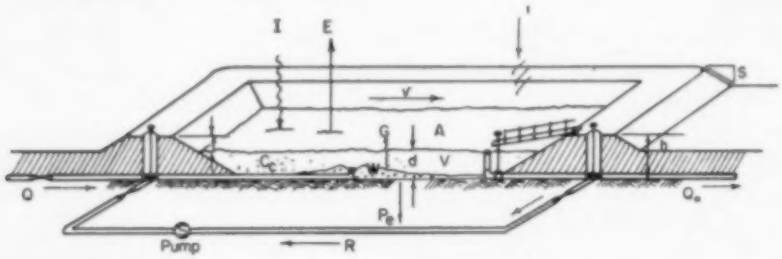
in the effluent is but 10 or 20 ppm, their presence will give rise to no difficulty, regardless of the point of discharge.

Unless short-circuiting of influent occurs, dissolved organic matter in pond effluents usually is highly stable and, therefore, the B.O.D. of the effluents is only from 20 to 50 ppm. A part of this B.O.D. may result from algal respiration if algae are present and incubation is in the dark. If incubation is in the light oxygen production by algae in the sample may result in an apparently negative B.O.D. As might be expected, the dark respiration of motile algae such as *Euglena* or *Chlamydomonas*, is much higher than that of the nonmotile types such as *Chlorella* or *Scenedesmus*. The respiratory requirement of algae is significant only if the effluent is discharged into waters where light, nutrient or temperature conditions may not permit photosynthesis. If algae are separated, B.O.D. of the effluent from properly designed ponds rarely exceeds 20 to 30 p.p.m.

The discharge of coliform organisms from ponds needs much additional study. Extreme variations are presently found in the published data (13) and these variations have not been systematically correlated with depth, detention period, arrangement of influent and effluent structures, series operation or other pond design factors. An unknown factor requiring early evaluation is the influence of chlorine on algae and bacteria in pond effluents.

PHYSICAL RELATIONSHIPS

A prerequisite for a complete discussion of the fundamental theory of ponds is the consideration of the dimensional factors involved and of their physical relationships. Fig. 1 schematically portrays a typical stabilization pond with one of many possible arrangements of influent and effluent structures. Letters indicate factors such as surface area and depth of the pond, while arrows are used to indicate processes such as evaporation, precipitation and percolation, processes which are of noteworthy importance in most ponds.



A	POND AREA	P_e	PERCOLATION
C_c	ALGAE	Q	QUANTITY OF INFLUENT
d	DEPTH	Q_o	QUANTITY OF OVERFLOW
E	EVAPORATION	R	RECIRCULATION RATE
f	FREEBOARD	S	SLOPE
G	GAS EVOLUTION	V	POND VOLUME
h	HEIGHT OF RETAINER	v	VELOCITY OF FLOW
I	LIGHT INTENSITY	W	WEIGHT OF SLUDGE
i	PRECIPITATION		

FIG. 1. Idealized projection of a stabilization pond showing design variables and symbols.

Based upon its length, l, width, w, and depth, d, a stabilization pond may be characterized by its surface area, $A = wl$, and volume, $V = wld$.

The quantity of waste, Q, entering a pond of fixed volume determines the approximate time of detention, D, for the wastes within the pond, thus:

$$D = V/Q, \tag{1}$$

in which Q and V are in the same volumetric units (e.g. V/Q, acre ft and Q, acre ft per day). More rigorously, influent, Q, effluent, Q_e, evaporation, E, precipitation, i, and percolation, P_e, must also be taken into consideration to compute the true value of D. Consider the total pond influent:

$$Q_i = (Q+i), \tag{2}$$

and the total pond effluent:

$$Q_e = (Q_e+E+P_e), \tag{3}$$

in which all values are expressed in volume units per day. If a pond attains equilibrium, by definition $Q_i = Q_e$ from which:

$$(Q+i) = (Q_e+E+P_e). \tag{4}$$

A molecule of water then would remain in the pond for an average period called the hydraulic detention period, D_h, defined as:

$$D_h = V/(Q+i) = V/(Q_e+E+P_e), \tag{5a}$$

or

$$D_h = V/Q_i = V/Q_e. \tag{5b}$$

Usually, it is sufficient to consider the hydraulic detention period as a function only of pond volume and waste flow, in which case equation (1) is applied. For ponds, the detention period is most conveniently expressed in days.

One of the essential physical conditions for the establishment of biological stabilization of wastes in ponds is the accumulation of a body of water of adequate depth, volume and surface area. Certain physical relationships must exist for this to occur. Essentially, three transient conditions require description.

(1) If upon initiating a pond, the amount of water lost through evaporation and percolation $(E+P_e)$ exceeds that of the influent $(Q+i)$, no water will accumulate in the pond. This condition indicates the existence of excessively porous bottom conditions or of excessive pond surface area, or of both.

(2) If $(E+P_e)$ is less than $(Q+i)$, water accumulates in the pond; but as depth increases, $(E+P_e)$ increases, and eventually at some depth attains a rate equal to $(Q+i)$, and an equilibrium depth becomes established in which no overflow leaves the pond. Many stabilization ponds attain equilibrium depth with no overflow, but this depth is usually considered inadequate if less than 3 ft.

(3) If $(Q+i)$ continues greater than $(E+P_e)$, depth increases until the discharge depth is reached and discharge of effluent occurs. At this time the volume of overflow is:

$$Q_o = (Q+i)-(E+P_e). \tag{6}$$

VOL. 5 1961

Inasmuch as evaporation, percolation, precipitation and sewage flow vary greatly with time and season, a pond may discharge effluent during one season and lose depth during another. Thus, for depth and detention period control, it is essential that each of these factors be considered in pond design.

The physical retention of organic matter in ponds also requires consideration. Soluble organic matter may leave the pond by way of overflow and percolation; but from the standpoint of receiving stream protection, overflow is of course more significant. Readily available dissolved organic matter is converted to insoluble material and settles to the bottom of the pond; but stable dissolved organic matter and finely dispersed suspended organic matter is discharged with the effluent. The resident time, D_s, for such soluble organic matter is:

$$D_s = V/(Q_e + P_e). \tag{7}$$

If $Q_0 = 0$, the theoretical resident time for soluble organic matter is V/P_e or V/Q. If $P_e = 0$ then $D_s = V/Q_e$.

The studies of Neel and Hopkins (15) indicate that the quantity of stable organic matter in the percolated water may be significant in certain cases. In their investigation they found that seepage from a pond carried detergents and, conceivably, other organic matter into a number of wells. However, it is unlikely that dissolved organic matter of the type readily available to bacteria will leave a pond through percolation, since it is usually converted into insoluble bacterial cells which would of course remain in a properly sealed pond until biological decomposition and dissolution occur.

LOADING PARAMETERS

Loading parameters commonly used for stabilization ponds are based upon population size, hydraulic loading and organic loading.

Loading on a population basis, N, persons per acre is:

$$N = P/A, \tag{8}$$

in which P is the equivalent population served, and A is the pond area in acres. The hydraulic loading is:

$$L_h = d/D, \tag{9}$$

in which d is the pond depth in feet or inches, and D is the average pond detention period in days (equation 1 or 5). The term d/D is called the hydraulic loading factor and is the rate of sewage application to a pond in feet or inches per day. The hydraulic loading parameter is especially useful because its units, length/time, are the same as those used for evaporation, percolation, and precipitation.

The organic load, L_o, applied to a pond in pounds per acre per day is:

$$L_o = k(d/D)(L_a), \tag{10}$$

in which L_a is the ultimate first stage B.O.D. of the waste in ppm and k is a constant having a value of 0·226 when converting ppm to lb per acre-in. per ppm and when

d is expressed in inches. The value for k is 2·71 when ppm are to be converted to lb per acre-ft per ppm and d is expressed in feet.

The actual B.O.D., L_t, to be satisfied in a pond is the difference between the ultimate influent B.O.D., L_a, and the ultimate effluent B.O.D., L_e, thus:

$$L_t = (L_a - L_e). \tag{11a}$$

Recognizing that both overflow and percolation may carry soluble B.O.D., whereas the water evaporated from a pond naturally contains little or no B.O.D., the exact quantity of oxygen or oxygen equivalent which must be supplied through pond resources is:

$$L_o' = kd\left(\frac{L_a}{D} - \frac{L_e}{D_s}\right), \tag{11b}$$

in which L_o' is the total oxygen demand to be supplied in the pond per acre-day and other factors are defined in equations (1), (7) and (10).

Before discussing in detail the magnitudes of the parameters set forth in the above equations, it is essential to consider in detail the several major reactions which normally occur in ponds and which are influenced by such parameters.

GENERAL REACTIONS OCCURRING IN PONDS

In rational pond design it is necessary to consider each fundamental purification process that takes place in the pond, its nature and mechanism, the organisms involved, its magnitude and importance in waste stabilization, its characteristic reaction rates, and the environmental and design factors which will either stimulate or retard its influence in this magnitude.

Types of decomposition in ponds

Fig. 2 shows schematically an overall view of the reactions which commonly occur when waste waters are continually added to ponds. Wastes that enter a pond undergo reactions which simultaneously bring about their deposition in the form

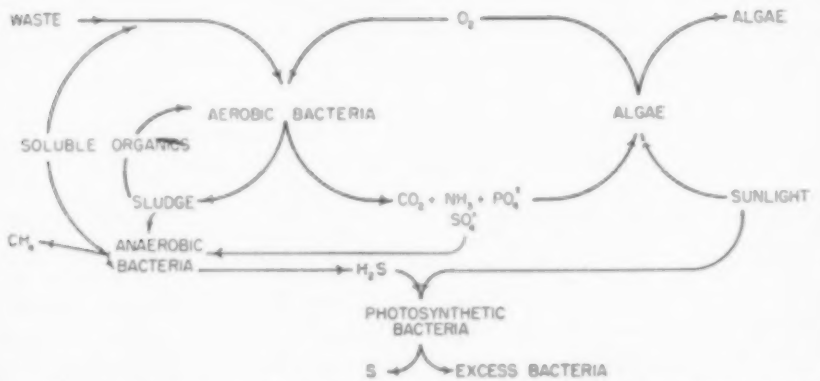


FIG. 2. Schematic representation of major biological cycles which may occur in stabilization ponds.

of sludge and their partial decomposition to carbon dioxide, ammonia, phosphate and other compounds. Once deposited, and if oxygen is available, sludge is decomposed aerobically and more of these compounds are formed. If little or no oxygen is available, sludge may undergo partial decomposition, may increase in quantity without decomposition, or it may undergo partial or complete anaerobic decomposition. Two types of anaerobic decomposition are involved. One form, sometimes known as putrefaction or acid-forming fermentation, utilizes oxygen from organic matter itself or from the oxygen-rich anions, such as sulfates, and gives rise to hydrogen, carbon dioxide, hydrogen sulfide and other odorous gases and to organic acids. The organic acids may accumulate in solution or serve as substrate for further breakdown. Under ideal conditions, the organic acid products of putrefaction quickly undergo a second type of fermentation, known as alkaline or methane fermentation, as a result of which methane gas is formed together with some carbon dioxide and hydrogen. An important decrease in pond oxygen resource requirements occurs when organic matter is decomposed through these fermentations.

When sunlight and nutrient are available, and time and temperature permit, green algae usually grow in ponds. The algae use the carbon dioxide, phosphate and ammonia resulting from bacterial decomposition to synthesize algal cell material and in so doing, release oxygen. This oxygen, produced in the surface waters of the pond, may, through mixing, become available for the oxidation of dissolved organic matter by bacteria. Thus, a cyclic process may become established in which unstable organic matter is converted to stable plant cell material, and the water is rendered free of unstable dissolved organic matter. In another process shown hydrogen sulfide may be converted to elemental sulfur by photosynthetic bacteria.

The accomplishment and control of the processes diagrammed in Fig. 2 constitute the fundamental objectives of design of stabilization ponds. To attain these objectives, specific levels of the various design parameters are selected that will bring about conditions favoring a desired group of biological organisms. Such conditions must be established and maintained for a sufficient period of time to permit the organisms to build a stable population. If extreme changes occur in the environment prior to establishment of a population, the desired processes simply will not take place. Moreover, the environment must be maintained within limits of variation to continue the process.

As a basis for recommending design parameters several of the major processes to be considered in more detail are: sludge deposition, aerobic oxidation, atmospheric reaeration, photosynthetic oxygenation, and anaerobic fermentation.

Sludge deposition

Sedimentation and bioflocculation are mainly responsible for the sludge deposition which occurs in ponds. However, "autoflocculation" and fecal deposition by invertebrates occasionally are significant.

Sedimentation and bioflocculation. In studies on physical sedimentation in pilot plant sedimentation ponds conducted by the University (16), it was found that

about 90 per cent of the suspended solids in raw sewage (mean temperature about 15°C) was removed within about 3 days, and 80 per cent of the dissolved solids in about 10 days. On the other hand, in ponds in which algal and bacterial growth had continued for some time, and in which the incoming waste water was mixed with the pond contents, 85 per cent of both the suspended and the dissolved solids were deposited at the bottom within 4 hr. From these results it may be concluded that bioflocculation brings about a ten-fold increase in the speed of formation and deposition of suspended solids, and a more than one hundred-fold increase in the speed of deposition of dissolved solids. The importance of bioflocculation and of design factors which lead to its occurrence is thus evident. The mechanisms of bioflocculation have been discussed at length by Lackey and Smith (17). It is believed by the author that their theories adequately describe the occurrence of the phenomenon as applied to stabilization ponds as well as in other types of treatment.

Other than a temperature above freezing, the essential conditions for the occurrence of bioflocculation appear to be the presence of a rich and varied population of micro-organisms, an ample supply of nutrients and sufficient time to develop a stable population. Bioflocculation is accelerated as temperature is increased from 4 to 25°C by the mixing action of waves or by mechanical recirculation and, probably, by the movements of invertebrates which sometimes inhabit the ponds. Although bioflocculation due to growths of facultative heterotrophs is known to occur under anaerobic conditions, the presence of oxygen greatly increases the incidence of bioflocculation.

VOL. 5 1961

Autoflocculation. Autoflocculation is a term applied to an interesting phenomenon which occurs under special conditions in ponds containing a dense algal population. As described in several recent reports (18, 19), autoflocculation occurs as the result of an increase in temperature and of the rise in pH level of pond waters which characterizes algal cultures during periods of vigorous photosynthesis. Certain compounds present in hard waste waters, such as magnesium hydroxide, calcium sulfate and ammonium calcium phosphate, become insoluble and are precipitated when the pH level and temperature in the pond are high. By enmeshing algal cells, detritus and bacteria, the precipitates form floc particles which readily settle. Coliform removal is known to be enormously enhanced by such autoflocculation. Autoflocculation is always followed by a great decrease in dissolved oxygen, because algae are carried into a zone of limited light and limited access to nutrients. Water softening associated for many years with algal growth in natural waters results from the extensive removal of calcium and magnesium ions in the radical autoflocculation of high rate ponds, and in some cases, as much as 50 per cent of the hardness may be removed (18).

Fecal deposition. Invertebrates such as *Rotifera*, *Cladocera*, *Ostracods* and *Copepods* are common in ponds, particularly during the spring and fall. These organisms ingest algae and bacteria, and are capable of rendering a pond virtually free of suspended solids, other than themselves, in a few days. However, only a small fraction of the bacteria and algae ingested are actually digested, the remainder are

simply added to the bottom sediments. Wennstrom (7), Uhlmann (20) and others attribute a considerable benefit to the overall purification in ponds from the proliferation of these organisms. A close scrutiny of their activity during field experiments has revealed that the spectacular clarification of pond contents results either from deposition of suspended matter in the form of their fecal pellets, or from bioflocculation and autoflocculation brought on by the gentle stirring action of their movements. Thus, only a fraction of the ingested material is stabilized by the organisms themselves while the remainder enters the benthal zone to be stabilized through anaerobic decomposition.

The magnitude of sludge deposition in a normally functioning pond is indicated by the data presented in Fig. 3. The figure shows the results of studies de-

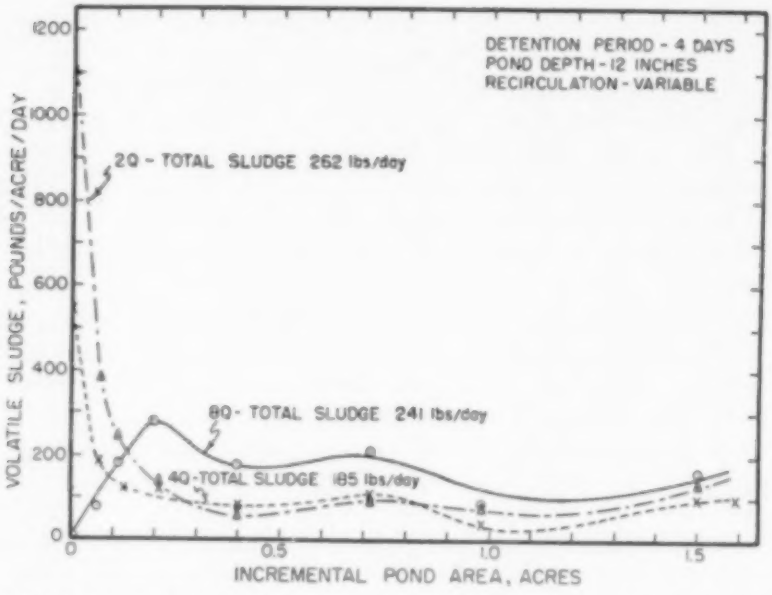


FIG. 3. The influence of recirculation on patterns of sludge deposition in a high rate stabilization pond.

signed to determine the distribution of settled solids which occurred in a high rate stabilization pond, the contents of which were subjected to various rates of recirculation (21). Low rates of recirculation were found to permit the deposition of large amounts of sludge near the pond influent structure, while higher recirculation rates distributed the sludge more widely. However, at each of the rates studied, some sludge was deposited over the entire pond area. The organic loading of the pond during the studies ranged from 90 to 120 lb of B.O.D. per acre per day. The daily deposition of sludge varied from 112 to 150 lb of volatile solids per acre per day. Assuming a B.O.D. to volatile solids ratio of 0·60, characteristic of the sewage

studied, it is evident that from 80 to 90 per cent of the organic content of each daily increment of sewage was deposited at the bottom of the pond.

Although the data presented in Fig. 3 do not entirely support such a conclusion, further experience has indicated that a great increase in the total deposition of sludge results from recirculation. This is to be expected since the seeding and oxygenation provided by the recirculant greatly increase bioflocculation.

The discussion on sludge deposition may be summed up as follows: in a well seeded, oxygenated and homogenized pond, from 80 to 90 per cent of the suspended and dissolved organic matter becomes fixed in organic solids, which settle to the pond bottom within a few hours. On the other hand, a pond which is not well mixed, oxygenated or seeded, acts merely as a sedimentation tank. In the latter pond, removal of solids is largely restricted to the settleable solids of the sewage, and takes place in proportion to the time of sedimentation. Dissolved solids tend to remain in solution. The presence of algae and invertebrates hastens sludge deposition through oxygenation, flocculation, mixing, ingestion, and defecation.

Aerobic oxidation

From the standpoint of odor nuisance prevention and complete organic stabilization, the organic matter contained in wastes being treated in stabilization ponds is best decomposed through aerobic oxidation. Although reports which list the specific organisms involved in aerobic oxidation in stabilization ponds are not available, it is extremely likely that the aerobic bacteria of ponds, which are mainly contained in a yellow-brown flocculent sludge (the substance created during bioflocculation) differ but little from those found in activated sludge or in trickling filter slimes (22).

VOL. 5 1961

Disregarding phosphorus, sulfur and trace elements, the oxidation of sewage organic matter in high rate ponds has been found experimentally (23) to follow the reaction:

$$C_{11}H_{29}O_7N + 14O_2 + H^+ \rightarrow 11CO_2 + 13H_2O + NH_4^+. \quad (12)$$

Oxidation to nitrate of the ammonia formed in equation (12) rarely occurs in stabilization ponds because ammonia is either assimilated by algae, lost to the air, or precipitated during periods of high pH before nitrification can become established. Thus, the weight of oxygen required to oxidize organic matter in a pond completely is shown by equation (14) to be about 1·56 times the weight of organic matter oxidized.

Encouragement of the process of aerobic oxidation of sewage requires that design parameters be selected which promote a large and varied bacterial population, a continued source of molecular oxygen, an ideal temperature, a uniformly optimum pH, a balanced nutrient source and adequate time for the process to become established. Although biological oxidation could ultimately result in the virtually complete oxidation of available organic matter and the removal of first stage B.O.D., it does not proceed to completion in ordinary ponds because the

sludge which reaches the pond bottom is in a zone usually void of molecular oxygen. Hence the deposited organic matter is no longer subject to aerobic oxidation and either must be decomposed anaerobically or must be resuspended for aerobic oxidation.

Inasmuch as the sewage which enters a stabilization pond is usually devoid of oxygen, only two sources of molecular oxygen are available for aerobic oxidation: atmospheric reaeration and photosynthesis.

Atmospheric reaeration

In the absence of green algae, under conditions which inhibit algal activity, or when a hydraulic detention period of but 3 to 5 days is used, continuous oxidation of organic matter causes the dissolved oxygen in a pond to fall below saturation at least during hours of darkness. A variable oxygen deficit will therefore develop in continuously and heavily loaded ponds. Depending upon the magnitude of the oxygen deficit, oxygen will diffuse through the pond surface, tending to restore the concentration of dissolved oxygen in the pond to saturation.

According to Hutchinson (24), however, molecular diffusion *per se* plays a negligible role in the reaeration of lakes. The reaeration values reported by Adeney and Becker (25) are ten-fold those obtained experimentally by investigators of lake reaeration. It has been recognized since the work of Adeney and Becker (25) that disturbance of the water surface and creation of new air–water surfaces is the key to accelerated atmospheric reaeration but a consistent pattern of surface agitation is difficult to maintain in ponds, hence reaeration also poses a problem.

No attempt is made in this article to employ the numerous formulations for atmospheric reaeration in order to predict its magnitude, particularly in view of the fact that such formulations do not apply to ponds which show an oxygen deficit at night and are supersaturated with oxygen during the day. However, for purposes of computations of oxygen balances in a later section of this article, the quantity of oxygen which will enter a pond by reaeration is assumed to be given by the approximate empirical equation of Imhoff and Fair (26):

$$R = 0{\cdot}0271(a)(d)(D_o), \tag{13}$$

in which R is reaeration in lb per acre per day, d is the pond depth in ft, D_o is the 24-hr average dissolved oxygen deficit, and the factor, a, is herein assigned an arbitrary value of about 20.

One reason that atmospheric reaeration is of little significance in pond design stems in part from the fact that the magnitude of oxygen deficit required to draw significant quantities of oxygen into a pond is that at which odors may arise. For example according to equation (13) a pond 3 ft deep and having an oxygen deficit of 10 ppm, will consistently gain only about 16 lb of oxygen per acre per day. Unfortunately a pond with a sustained oxygen deficit of 10 ppm would give rise to intolerable odors. In fact, observation indicates that an average oxygen deficit of 6 ppm is the maximum to be permitted, if odors are to be strictly avoided 24 hr per day.

If a deficit of but 6 ppm is permitted, only about 6 lb of oxygen per acre per day would be absorbed consistently in a pond 3 ft deep. As noted by Imhoff and Fair (26), these values are increased greatly by mixing brought about by wind action but it is not possible to depend on wind action. Ulhman (20) has suggested that the activities of invertebrates increase turbulence and hence, the uptake of oxygen. Caldwell (9) has pointed out, and it also has been observed by the author, that the surface of ponds approaching anaerobiosis become abnormally quiescent, and heavy wind action is required to generate ripples on the surface. Thus a pond which approaches septicity becomes more difficult to reaerate, and, consequently, the onset of complete anaerobiosis is hastened. On the other hand, a pond which contains from 5 to 10 ppm of dissolved oxygen will ripple in a slight breeze, and, consequently, may be more readily aerated than a pond on the verge of septicity.

A second reason that atmospheric reaeration is of little consequence in pond design is that less oxygen is taken up at night through reaeration than is lost during the day. In a pond supersaturated with dissolved oxygen during a part of the day, and having an appreciable concentration during the remainder, such as occurs in ponds having a large and active algal population the net tendency is for oxygen loss rather than for an oxygen gain. Hence, a dissolved oxygen barrier created through photosynthesis, in effect "excludes" aeration with atmospheric oxygen.

VOL. 5 1961

Oxygen production through photosynthesis

The major potential source of oxygen for aerobic oxidation in stabilization ponds results from algal photosynthesis. The classic reaction, $CO_2+2H_2O \rightarrow CH_2O+O_2+H_2O$, when modified to account for the usual composition of algae (23) may be rewritten as:

$$NH_4^+ + 7{\cdot}6\,CO_2 + 17{\cdot}7\,H_2O \rightarrow C_7H_{8{\cdot}1}O_{2{\cdot}5}N_{1{\cdot}0} + 7{\cdot}6\,O_2 + 15{\cdot}2\,H_2O + H^+ - 886 \text{ kcal.} \quad (14)$$

About 3·68 cal are fixed for each mg of oxygen liberated, and about 1·67 mg of oxygen are liberated for each mg of algae synthesized. The source of energy of this type of reaeration is, of course, the sun.

The types of algae most active in stabilization ponds are the microscopic Chlorophyceae, such as *Chlorella* and *Scenedesmus*, or as some cases, *Euglena* and *Chlamydomonas*. *Chlorella* and *Scenedesmus* are extremely hardy. Individual cells are found to be viable after long periods of anaerobiosis, drying and freezing. *Chlamydomonas* and *Euglena* on the other hand, die and decay readily under adverse conditions and hence may be less desirable in ponds which discharge in deep receiving bodies. *Chlamydomonas* may cover the pond surface and interfere with light penetration. Occasionally, duck weed or blue-green algae and diatoms of various types grow profusely in stabilization ponds (27). This may indicate some special water qualities or waste characteristics which inhibit green algal growth, but more probably indicates adverse design characteristics. The blue-green algae sometimes grow on sludge rafts at the surface and through their filaments lend added mechanical strength to the rafts. In other ponds blue-green algae have been observed to comprise the bulk of the suspended algal population (28).

The amount of oxygen production to be expected from a pond and the detention periods and depths which permit such production can be estimated by applying formulas derived by Oswald and Gotaas (11). These formulations, which require information on the overall photosynthetic efficiency, are reviewed in the following presentation.

The utilization of sunlight by algae to produce oxygen can be characterized by the photosynthetic efficiency, F, which is defined as the fraction of the available light energy converted to fixed energy in the form of algal cells, thus:

$$F = \frac{H}{E_s}, \tag{15}$$

in which H is the total energy fixed in algae and E_s is the total visible light energy penetrating the pond surface.

The detention period required in a pond for complete photosynthetic oxygenation of sewage is:

$$D = \frac{hC_c d'}{1000FS}, \tag{16a}$$

in which h is the specific heat of combustion of algae in cal/mg, C_c the algal concentration in mg/l., d' the pond depth in cm, F the conversion efficiency, D the detention period in days, and S the sunlight energy. By rewriting equation (16a) in terms of d'/D:

$$\frac{d'}{D} = \frac{1000FS}{hC_c}, \tag{16b}$$

and assuming $h = 6$ cal per day and converting d' in centimeters to d in inches we have:

$$\frac{d}{D} = 0{\cdot}66\frac{FS}{C_a}, \tag{17}$$

in which d/D is the loading factor, and F is expressed as a per cent. If it is assumed that conditions in the pond are such that all of the oxygen demand is to be met by photosynthetic oxygen production, and if the areal loading, L_o, is expressed in lb per acre per day, equations (10) and (17) may be combined to give:

$$L_o = 0{\cdot}226\frac{0{\cdot}66FS}{C_c}L_a. \tag{18}$$

Inasmuch as the ratio of oxygen to organic matter in photosynthesis (equation 14) is about 1·67, equation (18) may be expressed as:

$$L_o = 0{\cdot}25FS, \tag{19}$$

in which all terms are previously defined. Equation (19) shows that the permissible B.O.D. loading of a pond which obtains all of its oxygen from photosynthesis is a function only of the sunlight energy and of the photosynthetic efficiency attained

in the pond. Specific quantities of visible sunlight energy, S, in cal per cm² per day are presented in Table 1. By assuming various efficiency percentages, maximum and minimum rates of oxygen production as a function of latitude and month may be estimated from Table 1 using equation (19). It is interesting to note that if F is

TABLE 1. PROBABLE VALUES OF VISIBLE SOLAR ENERGY AS A FUNCTION OF LATITUDE AND MONTH

Latitude	Month											
	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
0 max	255*	266	271	266	249	236	238	252	269	265	256	253
min	210	219	206	188	182	103	137	167	207	203	202	195
10 max	223	244	264	271	270	262	265	266	266	248	228	225
min	179	184	193	183	192	129	158	176	196	181	176	162
20 max	183	213	246	271	284	284	282	272	252	224	190	182
min	134	140	168	170	194	148	172	177	176	150	138	120
30 max	136	176	218	261	290	296	289	271	231	192	148	126
min	76	96	134	151	184	163	178	166	147	113	90	70
40 max	80	130	181	241	286	298	288	258	203	152	95	66
min	30	53	95	125	162	173	172	147	112	72	42	24
50 max	28	70	141	210	271	297	280	236	166	100	40	26
min	10	19	58	97	144	176	155	125	73	40	15	7
60 max	7	32	107	176	249	294	268	205	126	43	10	5
min	2	4	33	79	132	174	144	100	38	26	3	1

* Values of S in cal per cm² per day.

To determine ave. value of S, S ave. $= S$ min $+ p$ (S max $- S$ min) in which p = total hours sunshine/total possible hours sunshine.

To determine yield of algal cell material, $Yc = 0{\cdot}15\ F\ S$ in lb of algal cell material per acre per day.

To determine yield of oxygen, $O_2 = 0{\cdot}25\ F\ S$ in lb of oxygen per acre per day.

After Oswald and Gotaas (11).

assumed to be 4·0, L_e will equal S. Therefore, when light conversion efficiency is 4 per cent the weight of oxygen produced in lb per acre per day is approximately equal to the daily insolation in Langeley's.

The photosynthetic or light conversion efficiency is a function of light, time, nutrient and temperature. Table 2 shows the results of a laboratory study made by the author (29) to determine the influence of these factors on light conversion efficiency. From the results of the study as presented in Table 2, an estimate of the probable light conversion efficiency may be made for a given set of assumed or locally established conditions which will compare well with values obtained in field studies. Thus, the data given in the table constitute a useful guide in determining the potential efficiencies attainable in high rate ponds when other conditions are

VOL. 5 1961

TABLE 2. PER CENT PHOTOSYNTHETIC EFFICIENCIES AS A FUNCTION OF ENVIRONMENTAL FACTORS

I Light energy flux S'		II Det. period D		III Diurnal illumination P		IV Applied B.O.D. L (5-day 20°C)		V Temp. T	
cal per day per l. $\times 10^{-3}$	F	days	F	per cent	F	ppm	F	°C	Tc
7·2	7·4	1·0	6·2	30	—	10	0·2	4	0·01
14·4	6·9	1·5	5·7	33	—	25	0·5	6	0·02
21·6	6·7	2·0	4·9	36	—	50	1·2	8	0·23
28·8	6·4	2·5	4·3	39	4·8	75	2·1	10	0.49
36·0	6·2	3·0	3·8	42	4·5	100	2·6	12	0·70
43·0	6·0	3·5	3·5	45	4·1	125	3·1	14	0·82
58·0	5·7	4·0	3·1	48	3·9	150	3·6	16	0·91
72·0	5·3	4·5	2·9	51	3·7	175	4·0	18	0·96
108·0	4·6	5·0	2·7	54	3·5	200	4·3	20	1·00
144·0	4·0	5·5	2·5	57	3·4	250	4·8	22	0·99
180·0	3·5	6·0	2·3	60	3·3	300	5·1	24	0·96
206·0	3·0	7·0	2·1	66	3·1	350	5·3	26	0·92
290·0	2·2	8·0	1·9	72	3·0	400	5·4	28	0·87
360·0	1·6	10·0	1·6	78	3·0	500	5·0	32	0·73

defined or assumed. The table lists various light intensities, detention periods, B.O.D. values, and per cent diurnal illumination, each with their corresponding efficiencies. These data are used in estimating efficiency according to the equation:

$$F = \frac{F_D + F'_S + F_L + F_P}{4} \times T_c \tag{20}$$

in which F is per cent efficiency, F_D the per cent efficiency listed in Table 2 as corresponding to the selected detention period, F'_S the corresponding efficiency for light expressed as cal/l. per day (Table 1, $S' = 1000\ S/d'$), F_P the corresponding efficiency for the period of diurnal illumination, F_L the corresponding efficiency for the 5-day B.O.D. applied to the pond, and T_c is the temperature coefficient as given in column V of Table 2. The resultant efficiency should be within the range of from 0·5 per cent to 6·0 per cent, and within 20 per cent of the actual per cent efficiency. Having determined the value of F, daily oxygen production can be computed according to equation (19), using the data listed in Table 1.

When estimating efficiency according to the method outlined, values for detention period, D, depth, d, and cell concentration, C_c, are required. Detention period may be computed according to equation (16), but this involves an assumed value of F, hence it is best to assume a value for D at the onset, and then to arrive at the exact value by successive approximations. The evaluation of C_c is based upon the ratio of oxygen production to algal cell material synthesized being about 1·67 accord-

ing to equation (14). If it is assumed that all of the oxygen produced through photosynthesis is available for bacterial oxidation, the value for C_c is:

$$C_c = \frac{L_t}{1{\cdot}67}, \tag{21}$$

in which L_t is the ultimate B.O.D. to be satisfied in the pond as estimated from equation (11).

On a practical basis as pointed out previously not all of the oxygen produced during photosynthesis will be available for oxidation. To arrive at an estimate of the extent of required oxygen production, a correction factor should be applied to account for the oxygen lost during the periods of oxygen supersaturation characteristic of ponds having an abundance of algae. Supersaturation occurs in such ponds during the time that photosynthesis takes place in the surface layers, and, in warm periods, due to decrease in water density, these oxygen-bearing layers tend to remain at the surface of the pond. Under such conditions, the dissolved oxygen concentration may increase to three-fold the saturation point affording an opportunity for loss of a considerable part of the excess oxygen through deaeration. Moreover, if the pond is discharging effluent some oxygen in the water is to be desired. Accordingly, the value of C_c as derived from equation (21) is multiplied by a correction factor termed the oxygenation factor O_f (30), which is the ratio of total oxygen produced to total oxygen required, thus:

$$C_c = \frac{L_t O_f}{1{\cdot}67}. \tag{22}$$

Evidence indicates that highest B.O.D. removal is obtained in ponds with oxygenation factors from 1·2 to 1·6 (30). At these values for the oxygenation factor, values for C_c would range from 0·72 to 0·92 that of L_t.

One evaluation of d rests on the fact that characteristically, algal cultures attain a cell concentration which tends to be inversely proportional to the depth of the culture. Experience has shown that in homogeneous cultures of algae, the concentration of algal cells tends to increase until light no longer penetrates to the bottom of the culture vessel. At this level lack of light tends to limit growth and the cell concentration tends to become stabilized. The relationship between algal concentration, light and depth is:

$$d' = \frac{\ln I_0 - \ln I'_d}{C_c \alpha}, \tag{23}$$

in which α is an absorption coefficient, I_0 the light intensity at the pond surface, I'_d the intensity at depth, d', and C_c the culture concentration derived from equation (22). The intensity, I_d, corresponds to the "compensation point" defined by Meyers (31) as the minimum intensity at which sufficient oxygen is produced through the photosynthetic activity of the algal cell to meet its respiratory requirements. In most cultures, the value for the compensation intensity is about 20 ft-c. The term α is a variable whose value increases with increasing pond depth because the increased

VOL. 5 1961

chlorophyll content of cells growing in light of the low intensity prevailing in the deeper reaches of a pond changes their light absorption characteristics. Data for light penetration and for α shown in Fig. 4 were obtained from laboratory studies of algal suspensions from ponds. Inasmuch as the experimentally applied light intensity,

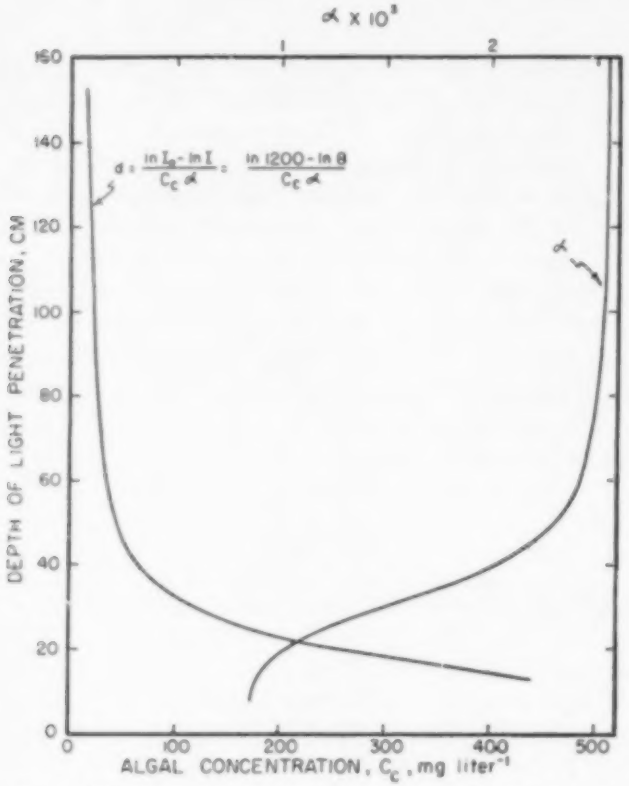


FIG. 4. Penetration of light of constant intensity through algal cultures of varying concentration and with varying light absorption characteristics.

I_0, in Fig. 4 was 1200 ft-c, a value of I_0 of 8 was used to make the numerator of equation (22) equal to 5·0, its average value for outdoor conditions. For reasons not completely understood, values for depths obtained according to equation (23) are considerably less than those which give highest photosynthetic efficiency.

Results of recent outdoor studies on the relation of depth to overall light energy conversion efficiency in continuously mixed ponds, indicate that the depth to be employed in pond design should be from one to three times that indicated by equation (23). Increasing the depth also has the added beneficial effect of discouraging emergent vegetation. With respect to the latter consideration, however, it should be recognized that the depth must be such that it be possible to maintain the algal

concentration at a level sufficiently great to meet the B.O.D. of the pond–waste mixture.

An anaerobic zone exists in ponds which are not mixed and which are deeper than the depth of light penetration indicated by equation (23). If the oxygen produced through photosynthesis is to be brought into contact with microbial cells in the presence of available organic matter which is found mainly at the pond bottom, mixing is essential in order that aerobic oxidation of the organic matter may occur. Mixing is accomplished to a varying degree in facultative ponds through action of wind, by convection currents, or by mechanical recirculation. Such methods are entirely inadequate for high rate ponds in which mechanical mixing is essential to the maintenance of aerobic conditions in the heavy sludge deposits characteristic of the process. Thus special apparatus for mechanical mixing must be included in high rate pond designs (19). If no mixing is provided or if the mixing is inadequate, aerobic oxidation is limited or precluded by lack of oxygen; and, therefore, unoxidized organic matter must either undergo storage or be decomposed anaerobically.

Sludge storage

Failure of sludge deposits to decompose usually is due to lack of suitable seed, low pH, the presence of inhibiting substances, or to a low temperature. When sludge is first deposited in newly initiated ponds, the amount of seed contained in it is inadequate for bringing about decomposition; and hence, time is required for the development of a sufficiently large population. Once a population is established, decomposition proceeds with the production of carbon dioxide, hydrogen sulfide, and organic acids. Frequently, accumulation of organic acids is so great as to cause a decrease in pH to a point below the optimum for bacterial activity. Substances that inhibit bacterial decomposition in ponds are strong acids, bases, and various organic and inorganic salts. The most common cause of sludge accumulation, however, is thermal inhibition. The rate of sludge decomposition declines with declining temperature, and practically ceases at 4°C. Under such conditions deposition exceeds decomposition, and undecomposed sludge accumulates until increased temperature conditions permit decomposition to begin. Nuisance seldom occurs in ponds at times when sludge decomposition is fully inhibited by freezing temperatures but it is a frequent problem during partial inhibition which is a common condition in temperate climates during the winter. Partial decomposition of accumulations of sludge may decrease the pH to a point where the material "pickles" itself and the pond may be said to be "stuck", much as a stuck digestor.

Acid forming decomposition

The uncontrolled occurrence of acid formation in heavy sludge deposits built up over a period of time is the primary cause of objectionable odors which may be emitted by stabilization ponds. In the reaction as described in several textbooks (32, 33), carbon dioxide, hydrogen, ammonia, organic acids, and many odorous compounds such as indole, skatol, cadaverin and hydrogen sulfide are produced

VOL. 5 1961

in solution. When such acid conditions exist in a pond some of the compounds, particularly gases, escape freely into the atmosphere, and once escaped, some may constitute a severe odor nuisance.

The odor of hydrogen sulfide in septic sewage usually can be detected in concentrations on the order of 1 ppm. The organic matter of sewage may contain from 5 to 20 ppm of organic sulfur, which, if converted entirely into hydrogen sulfide would have a concentration of 1 μ-mole/l. In spite of these concentrations, evolution of hydrogen sulfide ordinarily is not a problem in stabilization ponds either because the compound dissociates to hydrogen and hydrosulfide ions when placed in solutions of high pH or because it is oxidized in the presence of molecular oxygen. The hydrosulfide ion, of course, lacks an odor. In distilled water, a strong solution of hydrogen sulfide is almost entirely dissociated and odor-free at pH 8·5, whereas, it is only 50 per cent dissociated and will liberate odors at pH 7·0. According to Hutchinson (24), at pH values from 6 to 11 the basicity over 10^{-4} moles/l., the concentration of the hydrosulfide ion in natural waters may be approximated by the expression:

$$HS^- = B^+, \tag{24}$$

in which B^+ is the concentration of base in the pond. It is because of this high degree of dissociation that anaerobic ponds having a pH above 8·5 do not emit odors even when hydrosulfide is present in relatively large amounts. Of course in aerobic or facultative ponds a pH of 8·5 usually indicates the production of oxygen. The oxygen combines readily with hydrosulfide.

The decomposition of organic matter is not the sole source of hydrogen sulfide. Under extreme anaerobic conditions, the oxidation-reduction potential of the pond liquid may decrease and remain at a level at which the sulfate-reducing bacteria can become established and carry out a reaction in which sulfate serves as a hydrogen acceptor for bio-oxidation, and hydrogen sulfide is produced. Sulfate reduction is greatly stimulated by the presence of a high concentration of sulfate in the domestic water supply contributing to the sewage flow, and by a large concentration of organic matter such as accumulates during periods of thermal or pH inhibition. On the basis of evidence presented by Ohle (24), it seems likely that the activities of the sulfate reducers may continue to a limited extent during low temperatures and at low pH, thus aggravating objectionable odor conditions.

Nitrite and nitrate reduction is not of importance in ponds, because these anions ordinarily are not present in significant concentration in the domestic water supply, nor are they formed from ammonia in appreciable concentration in heavily loaded stabilization ponds. Moreover, if algae are present, they assimilate the ammonia before it can be oxidized. The application of nitrate to supply oxygen and raise the redox potential in industrial waste ponds has been recommended as a control of odors resulting from sulfate reduction. Practical experience has shown, however, that this chemical has very little effect on the production of odors in domestic sewage stabilization ponds. The best assurance against odors is maintenance of aerobic conditions, or loading at a rate sufficiently low to permit digestion of the influent organic matter.

Methane fermentation

Available information on the methane fermentation of sludge and of algae (34, 35) indicates that all of the materials which settle to the bottom of a stabilization pond through the various processes of sludge deposition are subject to methane fermentation, provided proper conditions exist or become established.

The well known essential conditions for methane fermentation are: an abundance of organic matter being continually converted to fermentable organic acids, an adequate population of methane bacteria, a pH level within the range of from 6·5 to 7·5 (preferably 7·0), alkalinity in quantities sufficient to buffer the organic acids, temperatures from 5°C to 60°C, the lack of toxic substances, and a sustained absence of oxygen (36). Methane fermentation is not always spontaneously established in the bottom deposits of a stabilization pond. In some cases the necessary environmental requirements may not be met or sustained a sufficient length of time, and may even be excluded by the design and operation of the pond. Should the required conditions exist in the sediments for a sufficient period of time, eventually methane fermentation will become established; and once established, it will contribute significantly to B.O.D. removal. Inasmuch as no quantitative data are to be found in the literature on methane fermentation in stabilization ponds, a study was made of the rate and composition of gas evolved from several stabilization ponds. A gas bell of known aperature was used to catch gas bubbles arising from the pond. Fig. 5 indicates in ft^3/acre/day the quantity of gas captured from a heavily loaded pond. Observations were hampered by complications arising from the high rate of recirculation used in the pond. The increase in quantity of gas as a function of distance from

VOL. 5 1961

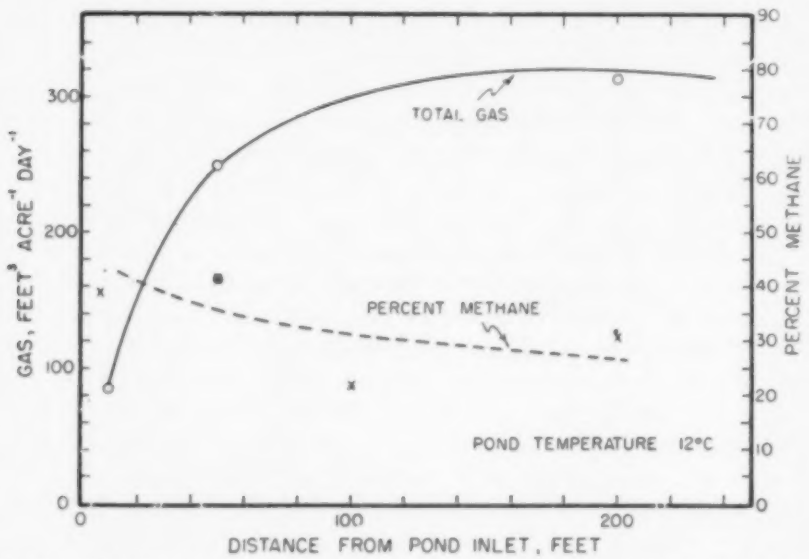


FIG. 5. Quantities of gas, and methane content of gas emitted from deposited sludge in a heavily loaded facultative stabilization pond.

the inlet resulted not from any time lag of sedimentation, but mainly from the transportation of the settling sludge away from the inlet by the high velocity created by inlet and recirculant flow and perhaps by the fact that oxygen in the recirculant hampered digestion. The variable fraction of methane shown in Fig. 5 as decreasing with distance from the inlet may have resulted from a difference in average temperature at the various positions or merely from variations in the amount of sludge. Although only average temperature data for the entire pond was obtained, it may be safely assumed that warmer sludge temperatures prevailed near the inlet. Experience indicates that the temperature optimum for acid formation within a pond is lower than that at which methane fermentation will take place. Inasmuch as hydrogen is formed by facultative organisms, it is to be expected that more hydrogen and less methane would be produced as temperature decreases.

TABLE 3. AVERAGE COMPOSITION OF GASES EVOLVED FROM THE BOTTOM OF A FACULTATIVE POND[a]

Gas	Percent by volume
Methane	39·0±12·0
Oxygen	1·0±0·5
Carbon Dioxide	2·0±0·5
Hydrogen and other combustibles	12·5±5·0
Unidentified	45·5±10·0
Total	100·0

[a] Pond Temperature 12°C.

In Table 3 are listed the average values for the composition of the gases collected from a pond at different stations. The fractions differ considerably from those of digester gas, probably because large amounts of carbon dioxide and hydrogen sulfide became dissolved in the overlying liquid, and therefore were not trapped in the bell. Methane and hydrogen are, however, relatively insoluble, and escape as bubbles. It is noteworthy that on the average, methane, hydrogen and other combustible gases comprise more than 50 per cent of the total gas produced. Assuming that each pound of volatile matter destroyed in anaerobic processes will give rise to about 15 ft³ of gases, and assuming, as indicated in equation (12), that the ratio of oxygen required to organic matter oxidized is 1·56; then the 315 ft³ of gas evolved per acre in the area away from the inlet indicates the dissipation of about 33 lb of B.O.D. per acre per day. It follows that the oxygen demand which would otherwise have to be met in the pond through reaeration or photosynthesis is diminished to about 1 lb for each 10 ft³ of gas evolved.

From the evidence presented, one may conclude that because of the strong tendency for sludge to settle in ponds, and the objectionable conditions that result when putrefaction of this sludge occurs, methane fermentation of the bottom sludge must be encouraged if the successful operation of both anaerobic and facul-

tative stabilization ponds is to be assured. A consideration of this fact together with the results of the sludge distribution studies shown in Fig. 3, and the well established requirements for methane fermentation brings to light several important facts regarding designs which may foster establishment of methane fermentation in stabilization ponds. In the first place, indiscriminate sludge distribution is objectionable. When sludge is widely distributed over the pond bottom through recirculation, a blanket of sludge only a few thousandths of an inch in thickness is deposited each day. Because of its wide distribution, the sludge is exposed to the capricious changes in environment characteristic of ponds, such as variation in pH, dissolved oxygen, and temperature. The continuous occurrence of such changes in the immediate vicinity of the thin sludge blanket makes it highly unlikely that methane fermentation would ever become established in the material. Moreover, it is equally unlikely that oxygen will reach this material for a period sufficiently sustained to permit the organisms of aerobic oxidation to become established, thus the only remaining alternative is putrefaction with attendant odor problems.

In the second place, if the rate of sludge deposition and accumulation exceeds that of depletion by digestion under the thermal conditions prevailing at the pond bottom, the sludge will become acid and putrefaction will take place.

On the other hand, if a sludge blanket is permitted to accumulate near the pond influent by limiting recirculation, dissolved oxygen will be absent, and due to its proximity to the entering sewage which is normally warm, a constant and relatively high temperature and a uniform pH will prevail. If such an area is seeded with methane producing organisms, methane fermentation will be initiated in the sludge mass and will continue providing loading is neither excessive nor too light. As a result a high B.O.D. removal will be obtained and the pond wil be free of odors.

VOL. 5 1961

POND CLASSIFICATIONS

Although the potential variations in pond classification are virtually endless, for purposes of further discussion it is advantageous to group all waste stabilization ponds into the three classes named in the introduction — anaerobic, facultative and aerobic, depending upon the biological activity in the pond responsible for the major fraction of the B.O.D. removal attained.

The characteristics of four operating ponds typical of these groups are given in Table 4, as well as other pertinent data, such as size, performance, types of organisms, and so on. The data in Table 4 were selected to include only those obtained during the months usually most critical for stabilization ponds in California, namely November and December.

Anaerobic ponds

A photograph of a typical anaerobic stabilization pond is shown in Fig. 6. The pond was designed to receive the wastes from a fat reduction (rendering) plant. Its performance characteristics are listed in Table 4. It is evident from the data listed in the table, which shows an organic load of 680 lb per acre per day, that only

TABLE. 4. PERFORMANCE CHARACTERISTICS FOR TYPICAL STABILIZATION PONDS

Waste	Units	Pond type			
		Anaerobic	Facultative		High rate
		Reduction plant	Domestic primary effluent	Domestic raw sewage	Domestic primary effluent
Area	acres	0·5	13·0	2·0	0·017
Depth	in.	96	36	50	12
Det. period	days	38	5·6	50	3
Month		12	11	12	11
Pond temp. (ave.)	°C	12·5	12·0	7	13
Hydraulic load	in. per day	2·5	6·5	1·0	4
Influent B.O.D.	ppm	1200	116	200	188
B.O.D. load	lb per acre day	680	170	60	170
Effluent B.O.D.	ppm	280	48	34	19
B.O.D. removal	per cent	77	59	82	90
B.O.D. removal	lb per acre day	520	99	37	153
Dissolved O_2 (ave.)	ppm	Nil	1·0	8·0	9·5
Gas production	cuft per acre day	2000[a]	315	300[a]	Nil
Photosynthetic organisms		Thiopedia rosea	Chlorella	Euglena	Scenedesmus
Algal conc.	ppm	Nil	10·0	50·0	101
pH (ave.)		7·7	7·3	7·6	8·0
Odor		Slight (urine)	H_2S	Nil	Nil

[a]Estimated on basis of bubble counts.

FIG. 6. Lagoon in which reduction plant waste is decomposed anaerobically with evolution of methane and other gases.

VOL.
5
1961

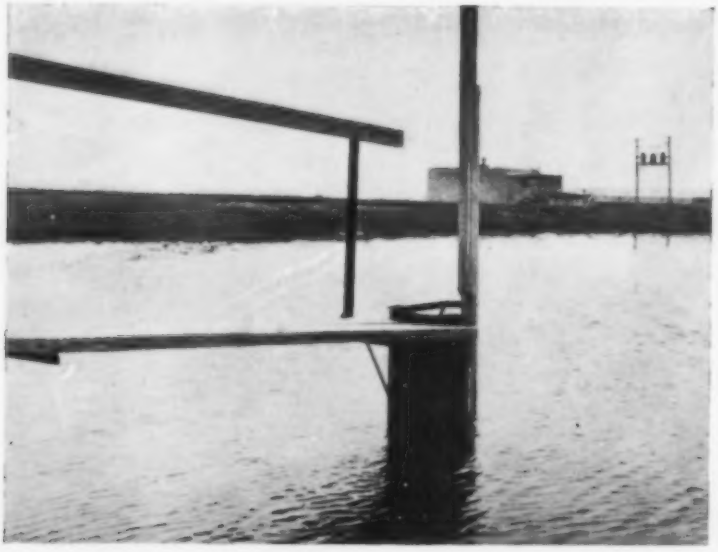


FIG. 7. A facultative raw sewage pond showing outlet structure, levees and surface appearance. Primary plant in background was by-passed to supply raw sewage to pond. (Courtesy of City of Woodland, Yolo County, California).

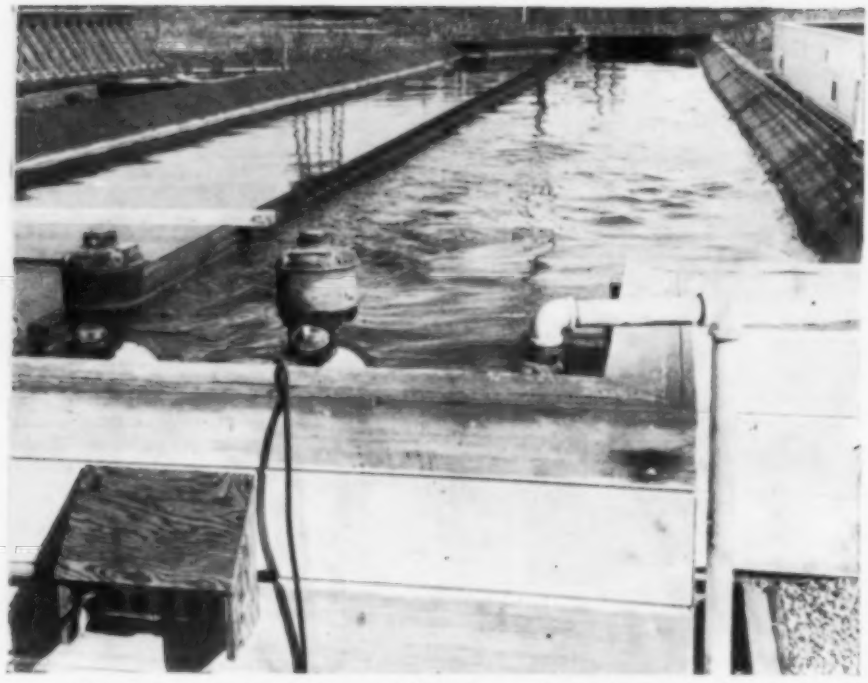


FIG. 8. A pilot plant high rate stabilization pond showing mixing facilities.

a pond undergoing active methane fermentation could accommodate such a load, maintain a pH of 7·7, and account for the reduction of B.O.D. attained.

Because it contains condenser water and hot wash water, the reduction plant waste has a mean temperature of about 40°C when it enters the pond through a 4 in. inlet pipe, which may be seen in the figure. The small surface area to volume ratio of the pond permits retention of much of the heat content of the waste; which in turn, greatly enhances digestion. The pond temperature varied between 12°C and 15°C during December, approximatley 5°C above that of its adjacent facultative pond, which was in thermal equilibrium with the environment. Observation showed that this anaerobic pond undergoes continuous vigorous mixing as a result of the rise of large masses of gas-impregnated sludge from the bottom. The upsurge of the masses accomplishes the mixing of the incoming warm water with the pond contents, and prevents direct short-circuiting of the waste. Rising organic matter establishes what appears to be an anaerobic bioflocculated sludge capable of absorbing incoming organic matter and settling readily. Dissolved organic matter is thus rendered insoluble and is carried into the zone of highly active sludge digestion. Digestion of the proteinaceous waste liberates ammonia leading to desirable alkaline conditions which stimulate digestion and prevent escape of hydrogen sulfide.

The supernatant from the pond is devoid of oxygen and is rich in soluble organic matter, ammonia and sulfides. Typically, the effluent B.O.D. is 250 ppm or more, a B.O.D. equivalent to moderately strong domestic sewage. It is evident, therefore, that the effluent would require additional treatment were it to be discharged into a water course. In the case of the pond described, however, the supernatant merely enters the adjacent facultative pond where it leaches into the surrounding soil.

VOL. 5 1961

Although anaerobic lagoons are frequently objectionable in appearance and often produce an odor nuisance, they cannot be challenged by other types of ponds with respect to efficient land use and total B.O.D. removal. It is interesting to note that an offensive odor is not an inevitable characteristic of an anaerobic lagoon, particularly if vigorous alkaline fermentation is in progress. But vaguely disagreeable odors are usually present, and it is for this reason that use of anaerobic lagoons should be restricted to isolated areas. Hydrogen sulfide, present at all times in large quantities in such lagoons, will be almost entirely in the form of the dissociated hydrosulfide ion in alkaline lagoons, and therefore is not usually liberated from solution. Hutchinson (24) states that such conditions probably provide optimum conditions for the development of sulfur bacteria. In the pond pictured in Fig. 6, *Thiopedia rosea* (37) completely covered the surface and converted hydrogen sulfide or hydrosulfide to sulfur almost as rapidly as it was formed, consequently further minimizing objectionable odors. When odors are at a minimum such ponds frequently sport a gaudy pink color and thus have been called "pink ponds".

Anaerobic lagoons should be designed to provide a small surface area to volume ratio so that maximum retention of the heat content of the waste can be attained. Evaporation is also decreased by a small surface area to volume ratios, a factor also conducive to heat conservation. A depth from 8 to 12 ft provides a good area-

volume ratio and allows for adequate sludge storage. A length–width ratio of 4:1 is beneficial, because it results in the deposition of a greater part of the sludge at the influent end. Consequently, the major digestive activity will take place in this region. Because less activity occurs near the effluent end, a greater degree of quiescence will prevail there, and the carry-over of floating organic matter to the secondary pond will be minimized.

As with separate sludge digesters, newly activated anaerobic lagoons require at least 3 to 6 months for natural alkaline fermentation to begin. Early overloading, lack of seed, production of excessive organic acids in the waste, sustained application of acid wastes, excess detergents, and low temperatures may prevent or retard the inception of alkaline fermentation beyond the normal period. If alkaline fermentation does not begin, the ponds become exceedingly offensive in appearance and odor. It naturally follows, as with digesters, that the onset of alkaline fermentation may be hastened in such ponds by massive seeding with vigorously digesting sludge from active ponds or digesters, and by maintenance of pH within the range of from 6·5 to 7·2 through judicious application of calcium hydroxide as suggested by Sawyer (38) for digesters.

From the preceding it would seem that great care must be exercised in initiation of digestion of acid wastes such as strong milk processing wastes or winery wastes to be treated in anaerobic stabilization ponds, because of the sensitivity of the methane organisms to pH levels below 6·5, and because of nuisance resulting from the evolution of hydrogen sulfide from acid waters. However, many dairy waste lagoons exist which apparently satisfy their owners and which appear to have a well established digestion (28). The role of sulfur bacteria in such lagoons requires much illumination.

The normally high B.O.D. of the effluent from anaerobic ponds requires controlled discharge to receiving waters or impoundment in facultative ponds of the type used for domestic sewage.

Facultative ponds

The facultative pond most closely represents the classic design described by Gillespie (4), Gieseke and Zeller (5), Caldwell (9), Hermann and Gloyna (12), and others. Presently in use throughout the country several variations of the pond type have resulted. In the west and south these ponds are usually designed as secondary treatment facilities and to have an overflow; while in the middle west and north the ponds as frequently are designed to receive raw sewage and to have no overflow. In the latter case evaporation and percolation usually are equal to the low rate of sewage application normally used.

The processes which occur in facultative ponds are sedimentation, bio-flocculation, aerobic oxidation, putrefaction, methane fermentation and photosynthesis. Inasmuch as oxygen usually is present in the pond supernatant, bioflocculation of incoming organic matter occurs rapidly. Because of the quiescence of such ponds, sedimentation of suspended solids quickly carries a large percentage of the organic

matter to the pond bottom. The loading and performance characteristics for two facultative ponds are shown in Table 4.

The pond described in Table 4 as receiving primary domestic sewage failed from overloading shortly after the data shown were taken and had to be drained because of hydrogen sulfide odors. It was loaded as heavily as 170 lb B.O.D. per acre per day, and was subjected to strong continuous recirculation. The pond apparently failed when methane fermentation ceased, because the heavy B.O.D. loading given it could only have been handled in such a pond by the combined action of atmospheric reaeration, photosynthetic oxygenation and methane fermentation. Although methane fermentation was well established in the pond in early November, reduced temperatures and possibly a reduction in pH resulting from reduced algal activity blocked continued fermentation later in the month. When drained, the pond bottom was found to be covered in places near the influent with a yellow-gray, foul smelling sludge having a pH of 5·9. When the pH of this sludge was increased to 7·5 by adding calcium hydroxide, it became gray and lost a great part of its vile odor at once, a common attribute of sewage sludge from "stuck" digesters. Exposed to the air, the sludge lost its odor in a few days. This experience clearly shows the problems resulting when one of the key processes for B.O.D. reduction in facultative ponds has become blocked by adverse environmental conditions and when other oxygen resources are not available.

VOL. 5 1961

The second facultative pond described in Table 4 is shown in Fig. 7. Raw sewage containing about 200 ppm of ultimate B.O.D. and an equivalent amount of suspended solids is applied to this pond at an average temperature-corrected loading of about 50 lb per acre per day. As is typical of properly designed ponds sight and odor nuisance are absent, and the pond water has a crisp and sparkling appearance. Algal concentration in the pond varies from time to time, normally averaging about 50 ppm. Bubbles of gas arising from the pond bottom indicates that vigorous fermentation is in progress.

The author has observed both directly and from descriptions in the literature (5, 6) that when first started in the fall, facultative stabilization ponds may initially become very odorous. The only biological activity that can occur in newly deposited sludge layers is putrefaction so that such odors tend to endure until methane fermentation becomes established, or until aerobic conditions exist throughout the pond depth. Aerobic conditions sometimes do not occur until the following spring when warmer temperatures permit accelerated algal growth and methane fermentation. Thus newly built ponds may undergo a period of several months or more of odor production before an ecological balance of oxidation and fermentation becomes established. In order to obviate such a delay, the pond shown in Fig. 7 was heavily seeded with algae and with digested sludge when initiated in the fall of 1959. Without suitable controls it is not possible to conclude that such seeding eliminated the typical starting problems, but it is worthy of note that the pond began gas production and attained saturation dissolved oxygen within 10 days after initiation and has remained highly aerobic since.

High rate ponds

In Fig. 8 is shown a pilot plant high rate pond in which mixing is supplied once per day to prevent deposited sludge from becoming anaerobic, and in which there is essentially complete dependence upon photosynthesis to furnish oxygen for the satisfaction of the applied B.O.D. Inasmuch as this type of pond has been described in previous publications (11, 30), only a brief *résumé* is required.

A high rate pond must be shallow enough to permit the entry of light almost to the bottom of the pond (see equation 23) and it must be mixed to maintain the deposited sludge in an aerobic condition. It must be lined to permit economical mixing to prevent undue turbidity and to control the growth of emergent vegetation.

Mixing is accomplished through the use of high-volume, low-head pumps, which are automatically turned on periodically to create a flow velocity in the pond liquid of about 1·5 ft per sec. This velocity supplies sufficient tractive force to suspend both deposited algae and deposited sludge, and to permit its contact with the oxygen rich supernatant at the pond. Experience has shown that a healthy aerobic sludge comparable to activated sludge is maintained in the pond, provided mixing is carried out for about 3 hr per day. Following an initial accumulation, the volume of aerobic sludge does not increase but rather remains constant, indicating essentially that total oxidation is taking place. During mixing, the sludge is suspended throughout the pond volume, but within 15 min after mixing ceases, more than 80 per cent of the sludge settles and is again distributed over the pond bottom so that sunlight entering the surface is not obscured. The algae do not adhere to this sludge nor do they become incorporated in it. Rather, they remain suspended to continue their synthesis in sunlight, unless an extremely high pH brings about autoflocculation. From the data presented in Table 4 it is evident that high rates of entirely odor-free B.O.D. removal may be attained in high rate ponds.

Oxygen balances

Oxygen balances are worked out in Table 5 for the four typical stabilization ponds presented in Table 4. In computing these oxygen balances, B.O.D. loadings were corrected to account for the reduction in biological activity resulting from the drop in ambient temperatures during the observational periods. Table 5 clearly shows a shift of importance from anaerobic to aerobic processes for B.O.D. removal as pond depth is decreased. Thus, in deep ponds anaerobic processes account for most of the B.O.D. removal; while in shallow, high rate ponds aerobic oxidation with oxygen produced through photosynthesis accounts for the primary oxidation. It is evident from the data listed in the table that photosynthetic oxygenation and methane fermentation are the key processes which make possible the reduction in B.O.D. accomplished in heavily loaded stabilization ponds.

TABLE 5. OXYGEN BALANCES FOR TYPICAL STABILIZATION PONDS

Pond type	Oxygen in lbs per acre per day			
	Anaerobic	Facultative		High rate
Waste	Reduction plant	Domestic primary effluent	Domestic raw sewage	Domestic primary effluent
O_2 equivalent of gas evolved from fermentation	330[a]	33	30[a]	Nil
O_2 from reaeration[b]	47	14	7	3[d]
O_2 from algal photosynthesis	Nil	23	15[c]	152
Total resources	377	70	52	155
Temperature corrected B.O.D. load	437	84	50	132
Net O_2	−60	−14	+2	+23

Estimated. (2) nocturnal reaeration:

[a] estimated on basis of bubble count

[b] estimated on basis of Equation 13

[c] estimated on basis of light penetration

[d] nocturnal reaeration

VOL. 5 1961

TABLE 6. SELECTION OF DESIGN PARAMETERS

Pond type	Basis for selection of detention period	Value for D in days	Basis for selection of depth	Value of d in ft
Anaerobic	digestive period at prevailing temperatures efficient land use	30–50	maintenance of anaerobic conditions, heat retention, evaporation control	8–10
Facultative Ponds with no overflow	evaporation and percolation	80–150	maintain conditions for digestion near inlet	3–5
Facultative Ponds with overflow	oxygen production bacterial die-away reaeration period	7–15 30–40 21–100	amount of algae required[a] weed control conditions for digestion near inlet	1–2 3–4 4–5
High-rate ponds	most efficient use of sunlight	2–6	penetration of light through active culture	0·6–1·0

[a] Paving of pond and mixing required.

SOME PRACTICAL POND DESIGN CONSIDERATIONS

Design criteria

Inasmuch as depth, detention period, and organic load established the major design criteria for ponds, values for these parameters are of interest. As indicated previously, four types of pond installations are available for engineering consideration. The data listed in Table 6 show ranges of values of both depth and detention period in the various pond types and a basis for the selection of depths and detention periods which will foster desired biological activities in the various types of ponds. Table 7 shows hydraulic and organic loads for the various pond types and the B.O.D. removal to be expected from each type. Selection of the type of pond to be applied in a specific circumstance is determined by waste and water quality, the receiving stream if any, climatological factors, the geographical distribution of population and by desired performance characteristics.

TABLE 7. HYDRAULIC AND ORGANIC LOADS AND B.O.D. REMOVAL FOR VARIOUS TYPES OF STABILIZATION PONDS

Type of pond	Hydraulic load in per day		B.O.D. load lb per acre day		Characteristic effluent B.O.D. data	
	occasional odors	without odor	occasional odors	without odor	ppm	per cent removal
Anaerobic	1–2[a]	—	300–500[a]	—	100–300	50–70
Facultative	1–3[b]	½–1	50–150[b]	20–50	20–50	70–85
High rate	—	2–5[b]	—	100–203[b]	20–30	80–95

[a] Assumes waste B.O.D. of 1200 ppm.
[b] Assumes waste B.O.D. of 200 ppm.

Multiple pond systems

Many designs will embody a number of separate ponds combined into pond systems. In considering such multiple pond systems it should be recognized that combinations of more than one of the types of ponds described in Tables 6 and 7 may be placed in series or parallel to give certain advantages which will be discussed.

In Fig. 9 is shown the various patterns in which a simple tetrad of four identical ponds may be loaded. As indicated the pattern of loading determines the type of pond and its performance. Although, a large number of series or parallel flow and recirculation patterns can be devised for multiple ponds, several basic principles should be considered in selecting a flow pattern suited to a specific need. Ponds that discharge effluents continually into smaller or intermittent streams should be multiple ponds, operated in series to assure the highest quality effluent possible. On the other hand, single ponds, or ponds operated in parallel, should either have no discharge other than evaporation or percolation, or should be discharged only under controlled conditions. Series loading is, of course, ideal for ponds which

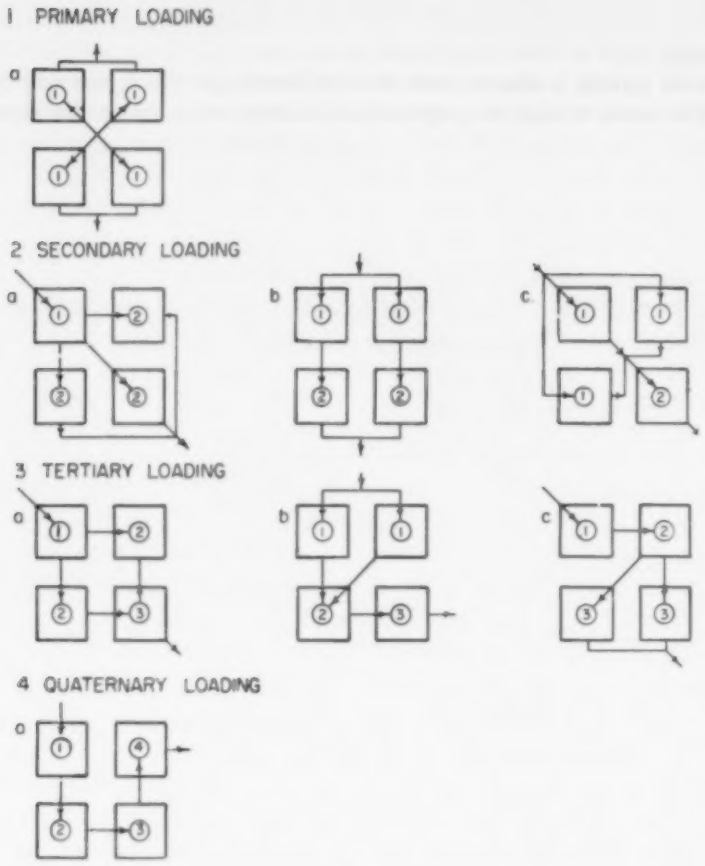


FIG. 9. Arrangement of various pond systems.

discharge effluent continually, because it gives the highest degree of solids, B.O.D., and coliform removal. Series loading is characterized by high B.O.D. loading in the primary pond, and low B.O.D. loading in the secondary pond. Anaerobiosis may occur in the primary pond, while aerobic conditions may frequently prevail in secondary, tertiary, or quaternary ponds. Recirculation of alkaline effluent waters may be employed to control odors in the primary anaerobic ponds, but indiscriminate application of recirculation may lead to conditions in the primary ponds which oppose the essential biological function of the pond. For example, during the winter recirculant has a much lower temperature than influent sewage, and therefore may increase thermal inhibition of methane fermentation in primary ponds.

Parallel loading is suitable mainly for ponds which have no discharge other than percolation or evaporation, which discharge to other ponds, or which discharge effluents only during the time when dilution water is available. A single pond that

VOL. 5 1961

discharges continually is subject to short-circuiting and carry-over of sewage solids containing high B.O.D. and high coliform densities, particularly during periods when wind mixing is absent, and the influent sewage is warmer than the pond contents. The warm sewage may spread in a layer over the cooler pond waters and may reach the pond outlet in the matter of minutes. Outlets may be designed to prevent this occurrence, but then sludge rafts may become a problem. Thus, in the event that coliform must be controlled in a receiving body of water, single and parallel ponds with overflows are to be avoided. On the other hand, by distributing an organic load equally among the available ponds, areal load is minimized, thus gaining the advantage of load distribution, seeding and pH control that recirculation yields, without the disadvantage of thermal inhibition.

Number, size and shape of ponds

In determining the proper number of ponds to construct for a given area, it is well to consider that a single pond gives maximum surface per unit of available area; whereas, the active area is reduced by multiple ponds, because a greater portion of the available area is devoted to earth work. If ponds are to be used solely as percolation and evaporation beds, cost is clearly minimized by minimizing their number. On the other hand, if flow patterns are considered essential to foster various biological processes, multiple ponds have evident advantages.

In general, the larger the pond the greater will be the amount of wind mixing and consequent homogenization of its contents. Serious levee damage has been observed by the author to occur in ponds of from 10 to 20 acres in extent. This can by prevented by rip-rap or bank pavement, but the additional cost may be greater than that of added levees. Moreover, if water penetrates beneath bank pavement it may undergo motion as a result of wave action. This motion increases in amplitude as the foundation gives way, and may ultimately lead to failure of the pavement and levee. As pond size is increased, freeboard must be added to contain large waves. Consequently, the height and cost of levees is amplified. One of the most serious problems in large ponds is that of floating sludge or algae rafts. The size and thickness of these increase with pond size.

A second problem with large ponds is seepage through the levees. In general, the larger the pond the less cut required to produce sufficient fill for levee material and the more the levees will project above the average ground surface. In such cases the line of ground saturation may intersect the surface above the levee base and excessive seepage may occur.

Little research has been carried out to determine the influence, if any, which pond shape may have on performance. Narrow inlets or bays in the ponds are, of course, undesirable, because stagnation may occur in these, leading to mosquito breeding and odor production. There is no evidence to oppose the practice of shaping ponds to fit the available area. Presently ponds are designed with level or slightly sloping bottoms to permit drainage. If sufficient area is available, square ponds with rounded

corners are best for parallel ponds; while elongated ponds with a length-width ratio of about two or three to one appear to be best for series ponds. The submerged central inlet apparently is the most satisfactory type for all varieties of ponds.

Future considerations

The facultative ponds described herein are ponds in which a combination of the three major processes, aerobic oxidation, fermentation, and photosynthesis are used together to obtain waste stabilization. Unfortunately, the compromises in environmental conditions required of each for the three to act together are so extreme, that none of the three processes may take place at a rate comparable to its true potential, hence the recommended loading for facultative ponds must be low.

The ponds described by Parker (6), in which anaerobic fermentation is fostered in shallow, heavily loaded initial ponds and photosynthetic oxygenation is fostered in secondary or tertiary ponds in series represent an important step toward the effective use of pond systems employing combinations of several types of ponds. A problem that arises from Parker's system is the odorous condition which he reports to exist at times in the initial ponds. It appears likely that increased depth for initial anaerobic ponds could improve digestion through preservation of heat and through increased sludge deposition. Improved digestion could, in turn, control odors as was outlined previously.

VOL. 5 1961

Efforts to control odors by recirculation of pond effluent are usually effective in lightly loaded ponds, but in heavily loaded ponds, as was pointed out previously, the physicochemical characteristics of the recirculant may retard methane fermentation needed to carry the load in the initial pond, and ultimately lead to failure. If the B.O.D. reducing capacity of anaerobic ponds could be combined with the odor controlling high pH and oxygen-rich conditions of aerobic ponds, the stabilization pond treatment process could be used more widely, because the areal requirement could be reduced greatly over that now required for facultative ponds.

A type of pond design which theoretically may have great potential is a deep, central pond surrounded and contiguous with a shallow pond of larger area. The wastes would be introduced into the deep central section in which methane fermentation should progress. In the peripheral shallow portions of the pond, photosynthesis should produce high pH, aerobic conditions, thus controlling odors. No systematic experimental work on ponds of this design has been carried out, but designs which are similar exist (39). On a theoretical basis, it appears likely that in moderate climates, if methane fermentation could be maintained, applied loads of several hundred pounds of B.O.D. per acre per day should be permissible in such ponds. However, caution is advised since allowable loadings and other design characteristics should receive extensive local experimental investigation before such ponds could be safely used in populated areas.

Another experimental concept in pond system design is that of physically separating the aerobic bacterial phase from the photosynthetic algal phase in an entirely aerobic system. It theoretically would be possible to design ponds which embody

a more favorable environment for each group of organisms than is available in current pond design. The concept of separating algal and bacterial phases has been termed "phase isolation". Through phase isolation B.O.D. loadings of 500 lb per acre per day are theoretically possible, providing sufficient light is available. Experimental work now in progress indicates that very dense concentrations of algae may be attained in growth units embodying the phase isolation principle (16), but pilot plant results are not sufficiently developed for inclusion of new design criteria at this time.

In addition to sewage treatment, use of dense algal cultures affords an opportunity for reclamation of nutrients. Concentrations of algae of the magnitude found in the phase isolation process appear to be within the range required for economic feasibility for separation. Once separated and dried, harvested algae comprise a by-product that will be valuable as an animal feedstuff (40), and which could offset a significant part of the cost of sewage treatment.

The production of maximum algal blooms in waste waters destined to enter lakes and reservoirs is of increasing importance. It is through the addition to lakes of carbonates, nitrates, phosphates and other compounds vital to algae, that nuisance algal blooms occur. Inasmuch as such compounds in sewage may be removed by dense cultures of algae it follows that high rate ponds should be considered as a means of removing nutrients wherever the need exists. Instead of serving as a possible support of nuisance blooms, these nutrient substances would be transformed into a potentially valuable by-product of the treatment process.

SUMMARY

The sequence of processes which take place in stabilization ponds are essentially identical to those employed in other forms of primary and secondary sewage treatment that occur in a vertical rather than horizontal sequence. As a result of coagulation, partial oxidation and bioflocculation, a major portion of solid, colloidal and dissolved organic and inorganic materials entering a pond rapidly settle and become a part of the bottom complex, where their decomposition is dependent on methane organisms.

The rates of bioflocculation and sedimentation in ponds are such that the supernatant rarely has a B.O.D. in excess of 50 ppm. Thus, a successfully operating pond acts as a diluting body of water, with reclaimed sewage comprising the dilution water.

Aerobic decomposition of bottom deposits in deep quiescent ponds is of no significance since the oxygen made available through reaeration and photosynthesis does not diffuse to the benthos. Decomposition of bottom deposits must, therefore, be accomplished through facultative decomposition and methane fermentation. In heavily loaded ponds, particularly during periods when methane fermentation either is non-existent or is limited by temperature, and algal photosynthesis is not taking place in the surface layers, a build-up of organic acid occurs with a subsequent lowering of the pH level. These conditions result in the emission of hydrogen sulfide from the pond and severe nuisance.

Even in lightly loaded deep ponds, sedimentation of organic matter is so complete that only a fraction of the B.O.D. applied has an opportunity to decompose aerobically, unless some form of mixing is used to maintain the organic matter in suspension. Intentional mixing of ponds, however, precludes methane fermentation, and hence imposes the requirement that the B.O.D. be met by atmospheric reaeration or photosynthesis. Photosynthesis at the rates required to oxygenate ponds completely produces concentrations of oxygen sufficiently high as to exclude reaeration. Thus, ponds designed for photosynthetic oxygenation exclusively must produce an excess of oxygen. Pond designs based on photosynthetic oxygenation also must provide for the collection and disposal of algae. If the algae are not removed, they will accumulate in large amounts. Eventually, the algae die and are decomposed with the result that objectionable conditions arise.

Ponds may be designed to encourage almost exclusively either methane fermentation (anaerobic ponds), or photosynthetic oxygenation (high rate ponds); and if successful, such ponds may convert several hundred pounds of B.O.D. per acre per day to methane or to carbon dioxide and ammonia, respectively. Ponds in which both photosynthetic oxygenation and methane fermentation occur simultaneously (facultative ponds) should be restricted to an average year-around design load to about 40 lb of B.O.D. per acre per day, if at times conditions occur which are unfavorable for either fermentation or photosynthesis. During the warm season, however, loadings two- or three-fold this average may be satisfactory on a short term basis.

Although ponds may seem extremely simple to the uninformed arbitrarily or carelessly executed pond designs, or designs which are excessively daring or excessively cautious may only be avoided through the recognition and application of the fundamental biological processes outlined here. The savings in public and private funds that will result from the skillful selection of design criteria suited to the employment of these processes at their full potential, constitute a worthwhile challenge to all waste control engineers.

Acknowledgements—I am indebted to Dr. Clarence G. Golueke and to Professor P. H. McGauhey for reviewing this manuscript. Portions of the author's research reported herein were supported in part by Research Grant 2601 C-7-9 from the National Institutes of Health of the United States Public Health Service.

REFERENCES

1. Edminston, F. E., *Fish Ponds for the Farm.* Chas. Scribner (1947).
2. von Schillinger, A., *Vom Wasser* **2**, 200 (1928).
3. von Merkel, Dr. (Editor), *Vom Wasser* **2**, 214 (1928).
4. Gillespie, C. G., *Sewage Wks. J.* **16**, 740 (1944).
5. Giesecke, F. E. and Zeller, P. J. A., *Eng. News Rec.* **117**, 674 (1936).
6. Parker C. D., Jones H. L., Taylor, W. S. and Greene, N. C., *Sewage Industr. Wastes* **22**, 760 (1950); **31**, 133 (1959).
7. Wennstrom, M., *Lund Universitets Arsskrift N.F.* **51**, 1 (1955).
8. Towne, W. W., Bartsch, A. F. and Davis, W. H., *Sewage Industr. Wastes* **29**, 377 (1957).
9. Caldwell, D. H., *Sewage Wks. J.* **18**, 433 (1946).
10. van Heuvelen, W. and Svorve, J. H., *Sewage Industr. Wastes* **26**, 771, (1954).

VOL. 5 1961

11. Oswald, W. J. and Gotaas, H. B., *Trans. Amer. Soc. Civ. E.* **122**, 73 (1947).
12. Hermann, E. R. and Gloyna, E. F., *Sewage Industr. Wastes* **30**, 511, 646, 693 (1958).
13. Fitzgerald, G. P. and Rohlich, G. A., *Sewage Industr. Wastes* **30**, 1213 (1958).
14. Amer. Soc. Civ. Eng., *Manual of Engineering Practice* **36**, 185 (1959).
15. Neel, J. K. and Hopkins, G. J., *Sewage and Industr. Wastes* **28**, 1326 (1956).
16. Oswald, W. J. and Golueke, C. G., Univ. of Calif. Inst Eng. Res., Series 149, 1 (1959).
17. Lackey, J. B. and Smith, D. B., *Biological Treatment of Sewage and Industrial Wastes*, vol. I, p. 108. Reinhold, New York (1956).
18. Oswald, W. J., Golueke, C. G. and Gee, H. M., Univ. of Calif. Water Resources Center Archives, Contribution 22, 1 (1959).
19. Oswald, W. J. and Golueke, C. G., *Advances in Applied Microbiology*, Vol. II, p. 292. Academic Press (1960).
20. von Uhlmann, D., Untersuchungen über die biologische Selbstreinigung häuslichen Abwassers in Teichen. *Wissenschaftliche Zeitschrift der Karl-Marx Universität*, Leipzig, 8 Jahrgang, 1958/59.
21. Oswald, W. J., Golueke, C. G. and Gotaas, H. B., *Studies of Algal Culture in a Field Scale Oxidation Pond*. Univ. of Calif. Inst. Eng. Res., Series 44, 10 (1959).
22. McKinney, R. E., *Biological Treatment of Sewage and Industrial Wastes*, vol. 1, p. 88. Reinhold, New York (1956).
23. Oswald, W. J., Hee, R. J. and Gotaas, H. B., *Studies of Photosynthetic Oxygenation*. Univ. of Calif. Inst. of Eng. Res., Series 44, 9 (1958).
24. Hutchinson, G. E., *A Treatise on Limnology*, vol I, pp. 586 and 759. Wiley, New York(1957).
25. Adney, W. E. and Becker, H. G., *Sci. Proc. R. Dublin Soc.* **15**, 385 (1918); **16**, 143 (1920).
26. Imhoff, K. and Fair, G. M., *Sewage Treatment*, p. 303. Wiley, New York (1940).
27. Nusbaum, I., *Trans. Amer. Soc. Civil Eng.* **122**, 98 (1957).
28. U.S. Public Health Service, Region VI, *Proceedings of Symposium on Waste Stabilization Lagoons, Kansas City, Missouri, August 1–5, 1960*.
29. Oswald, W. J., *Light Conversion Efficiency in Photosynthetic Oxygenation*. Thesis, Univ. of Calif., Berkeley (1957).
30. Myers, J., *Transactions of the Conference on the Use of Solar Energy*, vol. IV, p. 1. University of Arizona Press (1955).
31. Oswald, W. J., Gotaas, H. B., Golueke, C. G. and Kellen, W. R., *Sewage Industr. Wastes* **29**, 437 (1957).
32. Lackey, J. B. and Hendrickson, E. R., *Biological Treatment of Sewage and Industrial Wastes*, vol. II, p. 9. Reinhold, New York (1958).
33. Phelps, E. B., *Stream Sanitation*, p. 112. Wiley, New York (1944).
34. Golueke, C. G., *Sewage Industr. Wastes* **30**, 1225 (1959).
35. Golueke, C. G. and Oswald, W. J., *App. Microbiol.* **7**, 219 (1959).
36. Heukelekian, H., *Biological Treatment of Sewage and Industrial Wastes*, vol. II, p. 25. Reinhold New York (1958).
37. Lackey, J. B., Univ. of Florida, Personal communication (1959).
38. Sawyer, C. N., *Biological Treatment of Sewage and Industrial Wastes*, vol. II, p. 48. Reinhold, New York (1958).
39. Stone, R., *Civil Engineering* **30**, 3, 158–9 (1960).
40. Golueke, C. G., Oswald, W. J., Gee, H. K. and Cook, B. B., Production of low cost algal protein. *UNESCO International Symposium on Algal Culture, New Delhi, India, 1959*. Publication pending.

DISCUSSION

Werner N. Grune: You mentioned that there remain but two alternatives once acid fermentation becomes established in the sludge at the bottom of an oxidation pond: (a) drain the pond and remove the sludge, or (b) turn the pond into an activated sludge basin (presumably by aeration!)

What is your experience or have any efforts been made to try: (i) recirculation, or (ii) pH adjustment with lime (with and without recirculation)

AUTHOR: Recirculation has been tried without too much success. pH adjustment with lime has been considered, not tried thus far but is one of the things we shall try probably later this year.

VOL.
5
1961

ACCELERATED WASTE WATER OXIDATION POND PILOT PLANT STUDIES

NELSON L. NEMEROW

Syracuse University

A FULL-SCALED pilot plant consisting of five oxidation basins was designed, constructed, and operated during the first year of a two-year period from the summer of 1958 through the fall of 1959. Some effects of change in environment, loading, basin depth and close-baffled construction were ascertained during this period resulting in the following specific conclusions.

(1) No psychological nuisances were observed in the vicinity of the pilot plant during the study.

(2) Suspended solids removals were variable and generally similar to those obtained in primary settling basins.

(3) *Euglena*-type protozoans (or algae) were the predominant type of organism observed during the study.

(4) B.O.D. removals were greater in the fall and summer seasons than in the spring season. In fact, within the experimental loading limits (73–312 lb B.O.D./acre/day) environmental conditions appeared to have had more influence on basin efficiencies than B.O.D. loading (Fig. 1).

(5) The B.O.D. loading was most critical during the spring season. During this period the B.O.D. removals decreased with loading in 2-, 4-, and 8-ft deep basins (Fig. 1). The baffled 4-ft deep basin was not affected by B.O.D. loading up

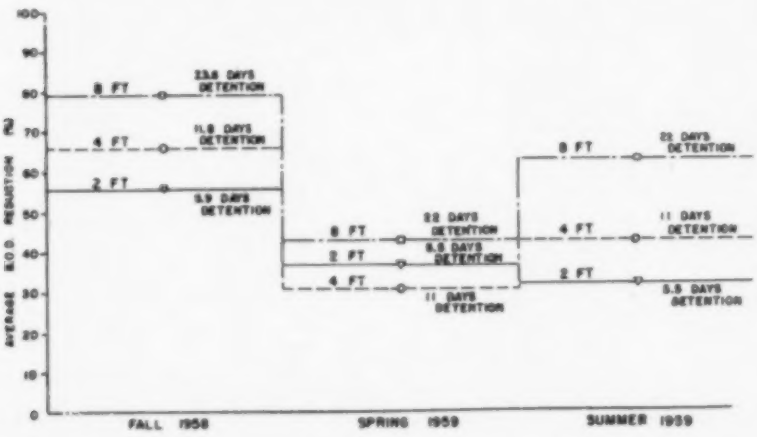


FIG. 1. Comparison of average basin efficiencies during the various seasons.

VOL. 5 1961

to 238 lb B.O.D./acre/day during this critical period. The average B.O.D. reductions during the entire year in all basins were as follows:

Basin No.	Depth (ft)	B.O.D. reduction avg. (%)
1	2	41·4
2	4 (baffled)	67·2
3	8	61·2
4	4 (constant loading)	56·9

(6) During the summer season at gradually increasing B.O.D. loadings of up to 1248 lb/acre/day only the 2-ft deep basin showed serious impairment. The peak reductions were obtained when loadings reached the 300–500 lb B.O.D./acre/day range. Average B.O.D. removals during the summer increased-loading study were as follows:

Basin No.	Depth (ft)	B.O.D. reduction avg. (%)
1	2	52·5
2	4 (baffled)	78·0
3	8	82·4
4	4 (constant loading)	69·2
5	4	54·2

(7) Basin efficiencies increased with depth (and detention time) during all seasons except the spring (Fig. 2). When comparing depth alone, the 8-ft deep tank perform-

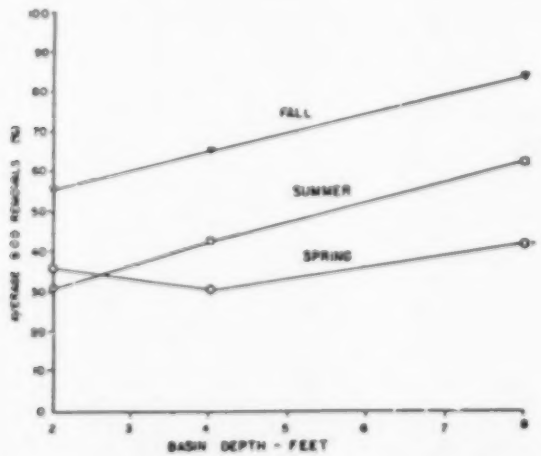


FIG. 2. Effect of basin depth upon average B.O.D. removals.

ed most efficiently during all seasons. A straight line relationship (about slope 4 : 1) existed between the B.O.D. removal in per cent and the depth in feet (between 2 and 8 ft) during the fall and summer seasons.

(8) During all seasons greater B.O.D. removals occurred in the baffled 4-ft deep basin than in the unbaffled basin of the same depth at similar loadings and detention periods. In addition, the 4-ft deep basin at constant loadings removed more B.O.D. than the same depth basin at increased loadings. The mean B.O.D. removals for the three 4-ft deep basins during each season were as follows:

Basin No.	Design Condition	Loading	B.O.D. Reduction		
			Fall %	Spring %	Summer %
2	Baffled	increasing	92·2*	52·8	73·3
4	Unbaffled	constant	78·0*	46·3	56·0
5	Unbaffled	increasing	80·0*	31·0	42·5

* Constant loading (135 lb B.O.D./acre/day).

(9) During the first yearly cycle of operation it appears safe to conclude that B.O.D. removals exceeding 80 per cent can be obtained in close-baffled 4-ft deep or 8-ft unbaffled basins during the critical summer period in Central New York State at elevated B.O.D. loadings of 312–467 lb/acre/day.

Acknowledgement—The financial support for a portion of this study of the New York State Health Department is gratefully appreciated. The technical assistance in encouraging the researcher and reviewing the results of Mr. A. F. Dappert, Mr. W. Larkin and Mr. R. Hennigan were very helpful in the overall study. The writer would also like to express his appreciation to the Onondaga Public Works Commission for its co-operation in carrying out the studies. Special thanks are given to Uhl T. Mann, Joseph Kieffer, and Alan Piontkowski of that Commission. The helpful participation of Mr. John Bryson in the studies of the fall of 1959 are appreciated.

VOL. 5 1961

OXYGEN ABSORPTION CAPACITY OF THE SIMPLEX HI-CONE AERATOR

E. Hurwitz, R. C. Gloppen and John Roeber

Current interest in "completely mixed" aerating systems for activated sludge treatment of strong industrial wastes prompted more intensive study of surface aeration devices and of combined surface aeration and sparger systems which are capable of producing "completely mixed" systems.

The efficiencies and aeration capacities of several systems that use combined surface plus sparger to accomplish aeration have been reported by Oldshue (1), Kalinske (2), and others.

There are several systems which use the principle of surface absorption to furnish oxygen to biological waste treatment systems. Among these the best known is perhaps the Simplex aerator which was developed in England and used extensively in the British Commonwealth and other countries. Its use in the United States has been slow in developing. There are many small installations but only recently has an interest been shown for application of surface aeration systems in larger plants.

VOL. 5 1961

Bolton and his associates (3) discussed the application of the Simplex aerator to the treatment of domestic waste or domestic waste in which a large portion of in-

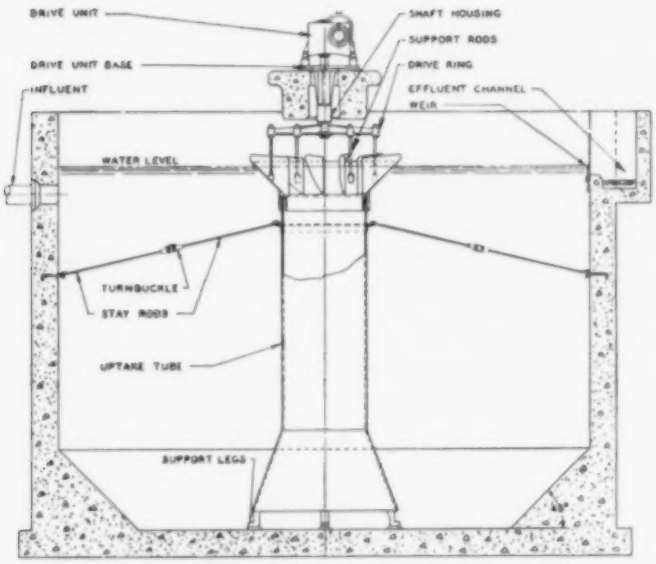


Fig. 1. Line drawing of a Hi-Cone surface aerator.

dustrial wastes are mixed. Studies of a large number of plants using aerating cones indicated that by increasing the speed of the cones, aeration periods could be decreased without affecting the degree of purification. However, there was a limit to the speed at which the original cones could be operated effectively and this led to the development of another type of aerating cone known as the High Intensity Aerating Cone (Fig. 1). This cone provided a higher rate of recirculation of the aeration tank content and a greatly increased intensity of aeration. Consequently the rate of purification was accelerated while the power demand was maintained at an economical level. Because the cones are adjusted to provide a constant rate of oxygen input, the intensity of aeration is directly related to the power input and can be conveniently expressed in terms of power input per unit volume of aeration tank.

Assuming that the reduction of B.O.D. in an activated sludge plant is a first order reaction, Bolton and associates computed the aeration periods necessary

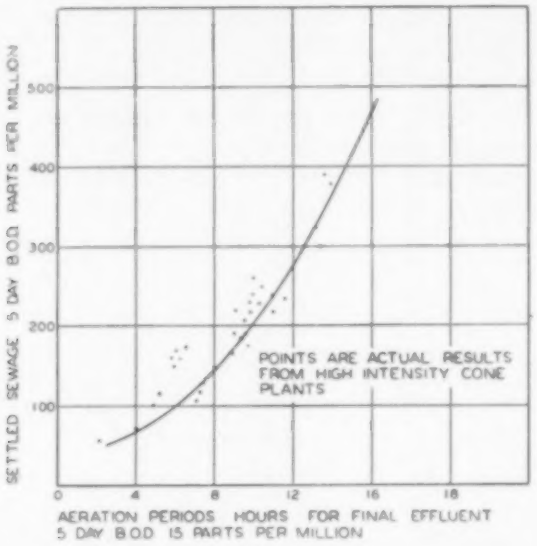


FIG. 2. Aeration periods required for complete purification with Simplex high intensity aerating cones.

Important: Aeration periods obtained from this graph give the total aeration tank volume required. No additional volume need be added for returned activated sludge.

to produce an effluent of 15 ppm from sewage of various B.O.D. concentrations and from these the power required per million gallons of sewage of the various B.O.D. concentrations. The results of their computations are shown in Figs. 2 and 3*. These curves were subsequently substantiated by studies at Manchester, England. The Manchester experiments also showed that treatment of the sewage of Manchester by surface aeration, using aerators of the Simplex type, had an advantage economically over diffused air bio-aeration Kessener and Pasveer systems.

* Points on Fig. 2 are actual results from a number of plants in normal operation.

Bolton and associates indicated that the intensity of aeration is a factor of the speed of the cone and the amount of cone exposure above water level.

In order to determine the effectiveness of surface aeration as accomplished by aeration devices of this type, oxygen absorption tests were made in the United

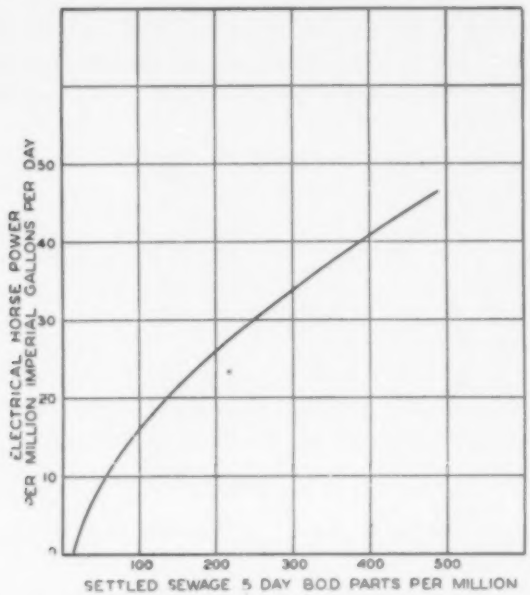


FIG. 3. Electrical horse power per million imperial gallons per day
Note: Curve indicates electrical energy supplied to motors.

States on a full scale tank 28 ft square and 13 ft 6 in. deep, with 3 ft, 45 degree bottom fillets. The cone used was a 6 ft high intensity (Hi-Cone) unit fitted with straightening vane to prevent vortexing in the draft tube. The aerator was kept in continuous operation between tests.

Oxygen absorption rate studies are generally made by one of two methods: (a) by diffusing air into a solution of sodium sulfite and measuring the rate of depletion of the sulfite and (b) by direct measurement of oxygen uptake in a suitable liquid media.

The sulfite method has been widely used and is simple to perform and to understand. However, Morgan and Bewtra(6) have shown that sulfite solutions are an "unnatural" media and that the rate of oxygen uptake is affected by such variables as sulfate accumulation, use or non-use of catalysts, and concentration of catalysts. They consider this method a useful and convenient scheme for creating non-standard conditions to simulate biological wastes having very high oxygen demand.

Morgan and Bewtra also consider direct measurement of oxygen absorption in biological suspensions an unsatisfactory method for comparing the oxygen uptake of different systems in different locations because of differences in the absorption media.

VOL. 5 1961

They conclude that the only reliable "standard" media for measuring oxygen uptake is clean tap water and that the variation in rate of oxygen uptake for varying water quality is well within the limits of accuracy of the test. De-aerated water was used as the media for measuring the oxygen absorption capacity of the Hi-Cone units in this test.

In aerating de-aerated water by means of surface absorption devices, the rate of absorption varies as the oxygen deficiency at atmospheric pressure. The relation of oxygen deficiency and the rate of oxygen absorption was formulated by Becker(9) and expressed as

$$K = (\log C_i - \log C_t)\frac{2.3}{t} \tag{1}$$

where K is the rate of oxygen absorption per unit time, expressed in ppm per ppm oxygen deficiency

C_i is the initial oxygen deficiency at zero time

C_t is the deficiency after time t.

The effect of temperature on the rate of oxygen absorption was shown by King[8] to vary as 1.024 T, where T is temperature in degrees Centigrade. The K value at the observed temperature is related to the K value at 20°C according to the equation:

$$K_{20} = K_t\, 1.024^{(20-T)} \tag{2}$$

The rates of oxygen absorption by the Hi-Cone unit used for this test are computed by use of the formula given above and corrected to 20°C.

TEST PROCEDURE

Measurements were made with the cone running at speeds of 25, 28.2, 30, 33, 42, 45, 48, 51, 54, 57 and 60 rev/min and with the lip of the cone at 1/2 in., 2 in. and 4 in. above the water level.

The water in the aeration tank was de-aerated by adding sodium sulfite in a slight excess of the amount required to satisfy the oxygen in solution. Cobalt was used as a catalyst. In the presence of the cobalt catalyst the oxidation rate of sulfite is extremely rapid. Therefore no appreciable error results from the presence of sulfite just before the oxygen uptake curve is started. However, because oxygen absorption is inversely proportional to the sulfate content, the number of runs completed with one tank of water was limited to avoid excess accumulation of sulfate. A maximum concentration of 1200 ppm seems indicated from available literature(5).

Samples were taken simultaneously from 12 stations in the tank at appropriate time intervals. The sampling points were at the midpoint of each wall at depths below the water surface of 1 ft, 6 ft 9 in., and 12 ft 6 in. Small submersible pumps were used as the sampling device.

Dissolved oxygen was determined by the azide modification of the Winkler Method(10).

RESULTS OF TESTS

The *K* value (oxygen absorbed per hour per ppm oxygen deficiency) at the observed temperature was determined from equation (1). This was corrected to 20°C by substituting in equation (2). From these *K* values the oxygen absorbed per hour at 20°C expressed in ppm are computed. These values are given in Table 1.

The weight of water contained in the tank at the 4 in., 2 in. and 1/2 in. cone exposures was 618,000 lb, 627,000 lb and 635,000 lb respectively. Power measure-

TABLE 1. OXYGEN ABSORPTION HI-CONE TEST

Inches cone exposure	rev/min	$K_{T°C}$	$K_{20°C}$	ppm O_2 per hr at 20°C	lb O_2 per hr at 20°C	net kW	net kWh per lb oxygen
2	25	0·86	0·96	8·8	5·5	1·85	0·335
2	28·2	1·29	1·57	14·4	9·0	2·4	0·267
2	30	1·73	1·95	17·9	11·2	2·7	0·241
2	33	1·76	2·52	23·1	14·5	3·1	0·214
2	36	2·15	3·13	28·7	18·0	3·5	0·196
2	39	3·34	3·74	34·3	21·5	4·0	0·186
2	42	3·96	4·36	39·9	25·0	4·5	0·180
2	45	4·47	4·92	45·1	28·3	5·1	0·180
2	48	5·03	5·54	50·7	31·8	5·6	0·176
2	51	5·54	6·10	55·8	35·0	6·4	0·183
2	54	6·10	6·70	61·4	38·5	7·2	0·190
2	57	6·65	7·30	67·0	42·0	8·3	0·197
2	60	7·18	7·90	72·5	45·5	9·5	0·209

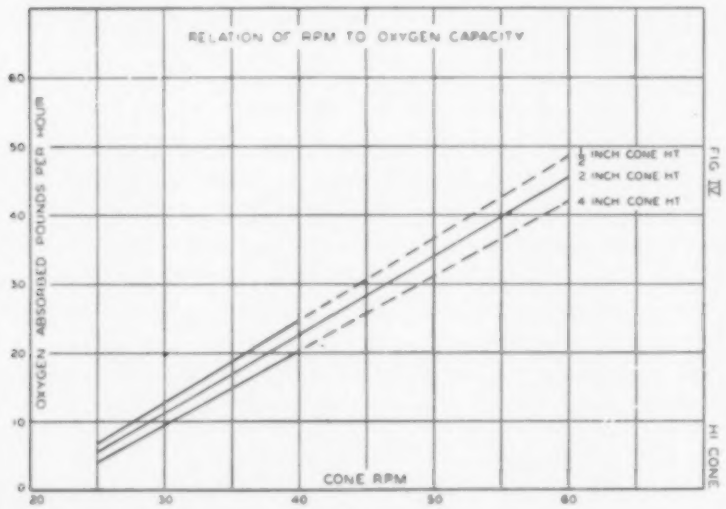


FIG. 4. Hi-Cone.

VOL. 5 1961

ments were made with a calibrated Weston analyzer and G.E. recording kilowatt meter.

The oxygenation capacities and power requirements of the aerator under the conditions of cone speed and cone exposure indicated are also given in Table 1 and are plotted in Figures 4 and 5.

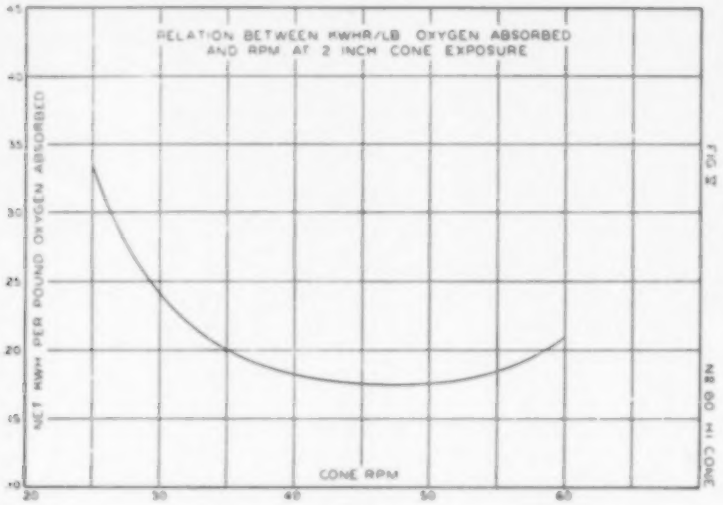


FIG. 5. No. 60 Hi-Cone.

The effectiveness of the Hi-Cone aerator is due, in part, to its action as an efficient low lift pump which maintains continuous circulation of the tank liquor. This circulation, in conjunction with the action of the cone blades, rapidly renews the air–water interfacial surface. The rate of turnover then becomes a significant factor affecting absorption capacity.

DISCUSSION OF RESULTS

Oxygen absorption in a Hi-Cone Aerator System varies directly as the cone speed (rev/min) and inversely as the cone exposure. There was a significant change in power consumption per pound of oxygen absorbed at the cone speeds tested. This varied from 0·34 kWh per pound of oxygen absorbed at 25 rev/min and cone exposure of 2 in., to 0·176 kWh per pound of oxygen absorbed at 48 rev/min.

The optimum power per pound of oxygen absorbed appears to be obtained at a cone speed of 48 rev/min at the 2 in. cone exposure in the unit tested.

The power measurements used in these data are net power. Corrections were made for the efficiencies of the motor and drive.

SUMMARY AND CONCLUSIONS

The tests indicate that at the 2 in. cone exposure and at the cone speeds indicated up to 45·5 lb of oxygen per hour were absorbed in a tank approximately 10.000 ft^3

in volume. This is at a rate of 4·5 lb per hour per 1000 ft³. Somewhat greater absorption is possible at the lower exposure (Fig. 4).

Power consumption varied from 0·34 kWh per pound of oxygen absorbed at 25 rev/min to 0·176 kWh at 48 rev/min.

REFERENCES

1. Oldshue J. Y., Aeration of biological systems using mixing impellers. *Biological Treatment of Sewage and Industrial Wastes*. Vol. I. Edited by McCabe and Eckenfelder. Reinhold.
2. Kalinske A. A., Mechanical air dispersion for sewage and waste treatment. *Biological Treatment of Sewage and Industrial Wastes*. Vol. I. Edited by McCabe and Eckenfelder. Reinhold.
3. Bolton J. F., Wishart J. M. and Sanderson F., Development of surface aeration with special reference to high intensity aeration technique. Conference on Biological Waste Treatment, Manhattan College, April 20–22, 1960.
4. McNicholas T. and Tench H. B., A review of recent activated sludge research at Manchester. Institute of Sewage Purification Annual Conference, 1959.
5. King H. R., Mechanics of oxygen absorption in spiral flow tanks. *Sewage and Industrial Wastes*. Vol. 27, Nos. 8, 9, 10 (1955).
6. Morgan P. F. and Bewtra J. K., Oxygenation efficiency in full-scale controlled aeration tank. Paper presented at the Fifteenth Annual Meeting of the Federation of Sewage and Industrial Wastes Association, October, 1959.
7. City of Manchester Rivers Department Reports, 1958 and 1959.
8. McNicholas J., Some trends in British sewage practice. Paper presented at 1959 meeting of the Canadian Institute on Sewage and Sanitation.
9. Becker H. G., Mechanism of absorption of moderately soluble gases in water. *Industr. Engng. Chem.* **16**, 1215 (1924).
10. *Standard Methods for the Examination of Water, Sewage and Industrial Wastes*. Tenth Edition. American Public Health Association, New York 19, New York.

VOL.
5
1961

DEVELOPMENTS IN BIOLOGICAL FILTRATION IN GREAT BRITAIN*

S. H. JENKINS and H. A. HAWKES

THE process of biological filtration is generally suitable for the climatic conditions in Great Britain since the winters, while cold, are rarely severe for long periods, the summers are not hot enough to result in serious odor nuisance being caused when the sewage is distributed on to a filter and, because of the smallness of the rivers and the high density of population and industry, effluents of good quality, nitrified if possible, are required. Since 1933 and especially in the post war period rapid progress has been made in the use of the activated sludge process in its various forms. However, developments have also taken place in the use of trickling filters. This paper is concerned with some of those developments in Great Britain.

VOL. 5 1961

RECIRCULATION

Recirculation was practised by Fowler and Holton (11) for the treatment of the strongly phenolic effluents from coal carbonization plants. The Birmingham Tame and Rea District Drainage Board installed recirculation in 1932 by re-cycling 16 mgd of filter bed effluent to the sedimentation tanks.

Operating four 120 ft diameter filters at Minworth Works, Mills (41) compared recirculation with single and also with alternating double filtration. Comparable effluents were produced when the recirculation filter treated the sewage at 150 gyd, the single filter at 75 gyd and the alternating double filters at 300 gyd. The recirculating filter gave trouble due to clogging of the upper layers of medium during winter. Tomlinson and Hall (50) confirmed these conclusions. Similar comparisons (46, 13, 52) showed recirculation to be less efficient than other methods tested. Barraclough (4) found recirculation superior to alternating filtration in treating the effluent from a high rate filter. From experiments at Sheffield, Edmondson and Goodrich (8) concluded that one filter operating on recirculation did the work of three single filters and gave an effluent with a lower B.O.D. but less nitrate. The efficiency of 4 ft deep filters treating sewage at Harrogate increased by 50 per cent when an equal volume of nitrified effluent was recycled (44).

The claims made for recirculation were investigated by Lumb (37) between 1946 and 1956. He concluded that the organic load could be applied at up to 2½ times normal loadings by using a 1:1 recirculation ratio. Except for the nitrate content, effluents of equal quality were obtained. As a result of further experience

* Abridged Paper.

Lumb and Eastwood (40) concluded that it was better to recirculate effluent in proportion to the flow of sewage or at a constant rate rather than to maintain a steady rate to the filters by reducing the volume of returned effluent as the sewage flow increased.

Roberts (49) used this last method of applying a steady flow at Leicester. From 1942–5 he applied loadings 2½ times as great as the load to a control filter with slightly superior results but with a reduced nitrate content. Besides the works mentioned above several new sewage works have made provision for recirculation of settled effluent, the larger of which are Keighley, Hereford, Swindon, Taunton and Cambridge.

Successful applications of recirculation have been made in industrial effluent treatment plants. Five volumes of effluent to one of cannery waste are used (7) for 12 hr daily during factory working hours and effluent only is circulated through the filters at other times. At the Monsanto Chemical Works, Ruabon, Wales (66), strong chemical wastes which may contain many substances used as pharmaceuticals or as rubber accelerators are diluted with weak liquors and corrected for pH. Figures for 1959 show an overall 91 per cent B.O.D. removal from a loading of 1.09 lb yd³ day, made up of 0.85 lb in the raw liquor and 0.24 lb in the recirculated effluent. Subsequent treatment by the surface aeration activated sludge process further reduces the B.O.D. to 6.1 ppm and sand filtration of this effluent lowers the suspended solid content and the B.O.D. to levels which do not support fungus growths in the receiving river. The nominal capital expenditure on this plant is approximately $ 1,400,000.

Table 1. Treatment of chemical works waste water at Fison's Pest Control Ltd., Harston, Cambridge (34)

	Raw liquor (ppm)	After adsorption with activated carbon (ppm)	Effluent after dilution, biological filtration, aeration and storage (ppm)
Phenols	800	8	0·8
Dinitro orthocresol	200	1	0·5
Color (as dinitrocresol)	500	7	3
Glycollic acid	1000	800	25
B.O.D.	2500	400	20

An interesting trickling filter plant for the treatment of chemical wastes is at the works of Fison's Pest Control Ltd., Harston, near Cambridge (34). Various pesticide intermediates and nitrophenols are present in the raw waste. Attempts to treat the effluent in admixture with sewage were unsuccessful owing to the bactericidal nature of some of the constituents. The waste is alkaline and it is therefore first neutralized with acid, then percolated over packed columns of activated carbon

in order to remove certain bactericidal phenols and dinitro orthocresol by adsorption. The carbon is reactivated by roasting in an oil-fired kiln at 900°C. The effluent is limed to precipitate copper and other salts, settled in a circular clarifier and the pH adjusted to neutrality. After dilution with river water (although effluent may equally well be used) settled sewage from the factory is added and the mixture treated on trickling filters. The filter effluent is passed to a concrete tank, aerated in one section of the tank, settled in another section and finally discharged to the river Cam. The results shown in Table 1 are typical.

It will be seen that the glycollic acid and other substances remaining after adsorption are oxidized during filtration.

THEORIES TO EXPLAIN IMPROVEMENTS BY RECIRCULATION

Wilson (66) suggests that the greater hydraulic loading may account for some of the improvements obtained with chemical works effluents, although Velz (57) regards the hydraulic loading of filters treating sewage as of little importance. Lumb (38) regards the presence of dissolved oxygen in a recirculatory system as important in maintaining aerobic conditions. Because of the cleaner condition of the filter he visualizes less oxygen being required for the biologically active growths, leaving more available for the oxidation of substances in the liquid. Lumb also discusses the suggestion that with a 1:1 recirculation ratio the volume of liquid available for carrying dissolved oxygen is doubled and the concentration of impurity nearly halved and since twice the work of a conventional filter is done, at least four times the amount of oxygen must be supplied, according to the Law of Mass Action. A third factor contributing to the added efficiency obtained with recirculation may be the reduced amount of growth in the upper layers of medium due to the reduced concentration of organic impurity, since Tomlinson (54) found that the growth of film with a 1:1 mixture of sewage and effluent was about one-fifth of the amount obtained with undiluted sewage. A more equable temperature within the bed and continuous seeding with aerobic organisms have also been suggested (38) as additional beneficial factors although lower winter temperatures at the surface of a filter have been noted in a filter operating with recirculation. Lumb also refers to the advantage of providing a greater hydraulic loading so as to give a better flushing action, allowing more use to be made of the available media surface, extending the zone of clarification and carbonaceous oxidation further down the bed though with the resultant reduction in nitrification. Belgian workers (35) have suggested that nitrite is more important than nitrate by being produced throughout purification and utilized in the deamination of amino compounds.

Activated sludge treatment as an aid to biological filtration

By pre-treatment of settled sewage with activated sludge for a contact period of 1 hr, sufficient flocculation occurs to allow the volumetric load applied to the filters to be increased without causing clogging. One bio-flocculation plant to deal with 11 mgd was built at Minworth in the period 1923–8 and another, handling 15 mgd, in 1930 (58, 32).

VOL. 5 1961

Alternating double filtration

In 1931 O' Shaughnessy at Birmingham (45) reported that when the effluent from one of the bioflocculation plants mentioned above was applied to a filter clogged with growth the obstructive matter disappeared and the filter could be re-used. He suggested operating a pair of filters in series until the first became obstructed with growth and then clearing it by dosing it with effluent from the second filter, which was then dealing with the sewage. Excellent results were obtained by the Water Pollution Research Board in large scale experiments with milk wastes (28, 59), using a pair of filters in series and alternating them periodically as primary and secondary filters. It had already been shown in laboratory experiments (29) that by interchanging the top and bottom sections of a trickling filter an overall increase in efficiency occurred. Several large plants dealing with milk wastes were built and, as the results summarized in Table 2 show, have given excellent results (43).

Table 2. Treatment of milk factory wastes (after Muers)

	Dairy			
	Ellesmere	Whitland	Bailey Gate	Chard
Vol. waste per day (galls)	84,000	168,000	192,000	66,000
B.O.D: of waste (ppm)	2000	250	550	700
Dilution ratio before filtration	1:2 or more	1:1	1:1	1:3
B.O.D. of effluent (ppm)	20	3	6	5

The results obtained by the Water Pollution Research Board with sewage under various experimental conditions using alternating filtration between 1940 and 1959 are shown in Fig. 1.

In the above investigations on the treatment of Minworth (Birmingham) sewage, Mills (42) and later Tomlinson and Hall (56) found that alternating double filtration was a more efficient process than single filtration or recirculation. Similar conclusions were reached by Greene (13) in experimental work with Derby sewage which consists of 60 per cent domestic sewage and 40 per cent of effluent from the manufacture of chemicals, textiles and plastics. On the basis of his experiments a plant to deal with 22 mgd has been constructed.

At Reading comparisons between single filtration, recirculation and alternating double filtration for the treatment of strong, industrial sewage showed (4) that alternating filtration best prevented clogging although recirculation gave superior effluents.

Tidswell (52) experimented on the treatment of sewage from Burton-on-Trent which has a population of 50,000 and an additional population equivalent of 200,000 as brewery effluent. He compared single filtration, recirculation, double filtration and alternating double filtration, activated sludge treatment having been rejected as a possible process as a result of earlier work. Double filtration and alternating

filtration were almost equally superior to the other processes. Double filtration without alternation produced a more highly nitrified effluent.

Alternating double filtration has been adopted for a $45 million improvement scheme at Birmingham and at several smaller plants such as Bedford, Caldercruix

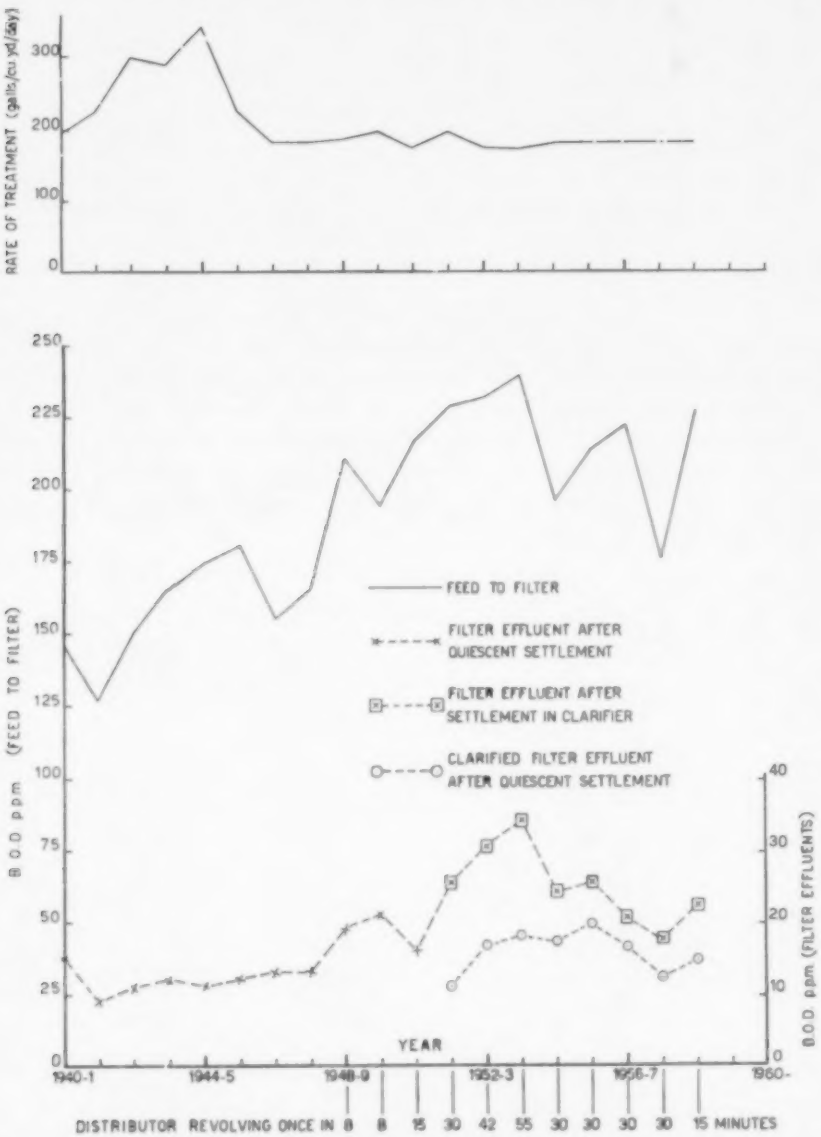


FIG. 1. Experiments on alternating double filtration at Minworth, Birmingham, using 120 ft diameter filters, from 1940–1959. The distributors were self-propelled until 1948. Thereafter they were driven at the speeds indicated. (From results obtained by the Water Pollution Research Laboratory and The Birmingham Tame and Rea District Drainage Board).

VOL. 5 1961

(Lanark), Havant, Blithe Valley Works (Stoke-on-Trent) and Derby. The results from Bedford, a typical plant, are given below (63).

Table 3. Results of treatment of sewage by alternating double filtration at Bedford, population 57,580 at 163 gyd (63)

B.O.D.: Settled sewage	236 ppm
Settled final effluent	6 ppm
Ammoniacal N in settled sewage	28 ppm
Nitrite plus Nitrate N in effluent	23 ppm
B.O.D. loading removed by primary filters	0·58 lb/yd³-day
B.O.D. loading removed by secondary filters	0·06 lb/yd³-day

THE "MECHANISM" OF ALTERNATING DOUBLE FILTRATION

Tomlinson (53) sought to explain the high efficency obtained by this process by investigating the rate of growth of the surface film. Filter medium placed in wire baskets at and below surface layer was taken out at intervals and the organic film, removed by washing, was examined chemically and biologically. The results shown in Fig. 2 demonstrate that the amount of growth per unit volume of medium was much less with alternating filtration than with single filtration. Since the load treated per unit volume was much greater with alternating filtration it is evident the film was more efficient.

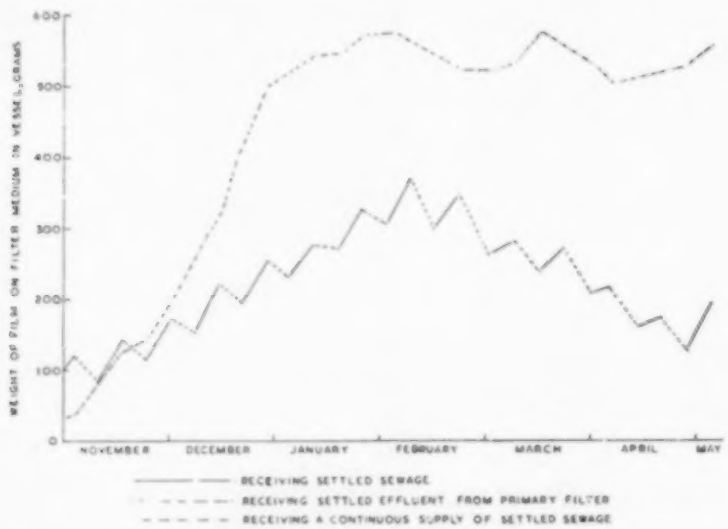


Fig. 2. Comparison between the amount of film present in unit volume of medium in a single filter receiving a continuous supply of settled sewage and in filters operated by alternating double filtration. In the latter case the amount of growth rises and falls as the filter alternately receives settled sewage and settled bed effluent from the primary filter. (From Tomlinson 1941 (53)

Microscopic examination of glass slides placed at the surface showed that when filter effluent was applied to a filter in the secondary stage the contents of some fungal cells underwent lysis, resulting in disintegration of the film. The alternating filters were found to give a steady discharge of humus instead of the periodic heavy unloading common with single filters. Figure 3 shows that growth of film was more marked in winter with the single filter than with alternating filters. Much of this growth was due to the fungus *Sepedonium*.

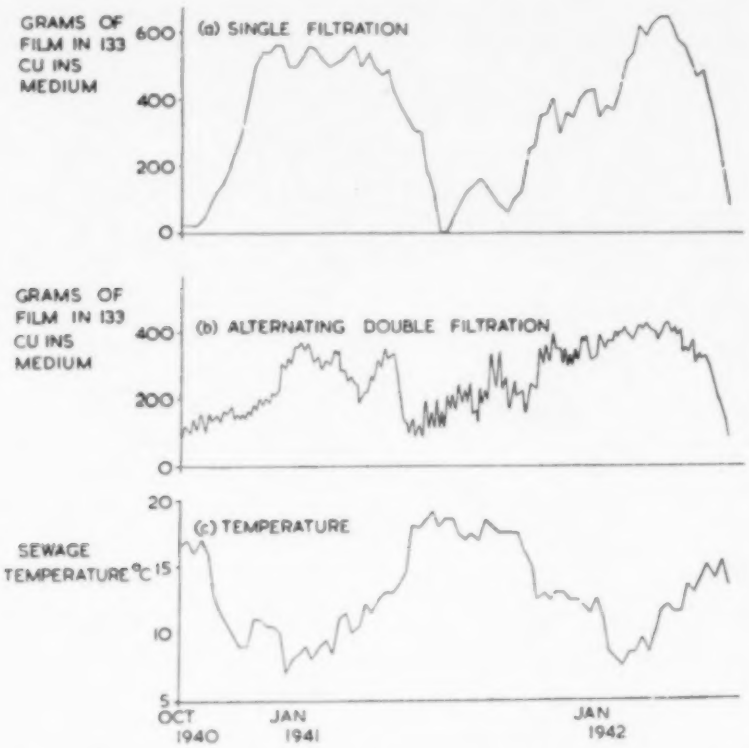


FIG. 3. Comparison between the seasonal amount of film in unit volume of medium in a single filter treating settled sewage and a filter operating by alternating double filtration. (From a paper by Tomlinson (54).

Double filtration

Double filtration without alternation has been practised for many years on sites where sewage could gravitate through the works, as at Chertsey (51) where treatment on the primary filters was at twice the rate used on the secondary, or at Lincoln (2) where settlement between the two stages was omitted. At Staines (14) sewage filtered at 300 gyd through clinker was then treated on fine gravel beds at about 100 gyd.

VOL. 5 1961

Intensive work on double filtration was carried out at Huddersfield (12) in two semi-scale filters each 18 ft diameter. The primary filter treated settled sewage at 750 gyd. After settlement the effluent was applied to the secondary filter at 360 gyd. The results were more consistently satisfactory than by single filtration at 110 gyd. After the outbreak of war in 1939 the sewage became increasingly toxic and these rates had to be reduced to give satisfactory results.

In connection with the investigations at Huddersfield, Reynoldson (47, 48) made ecological studies on the two experimental filters and on the large scale works filters.

At Minworth, Birmingham, double filtration at 600 gyd (overall rate 300 gyd) gave the same results as alternating filtration and recirculation between March

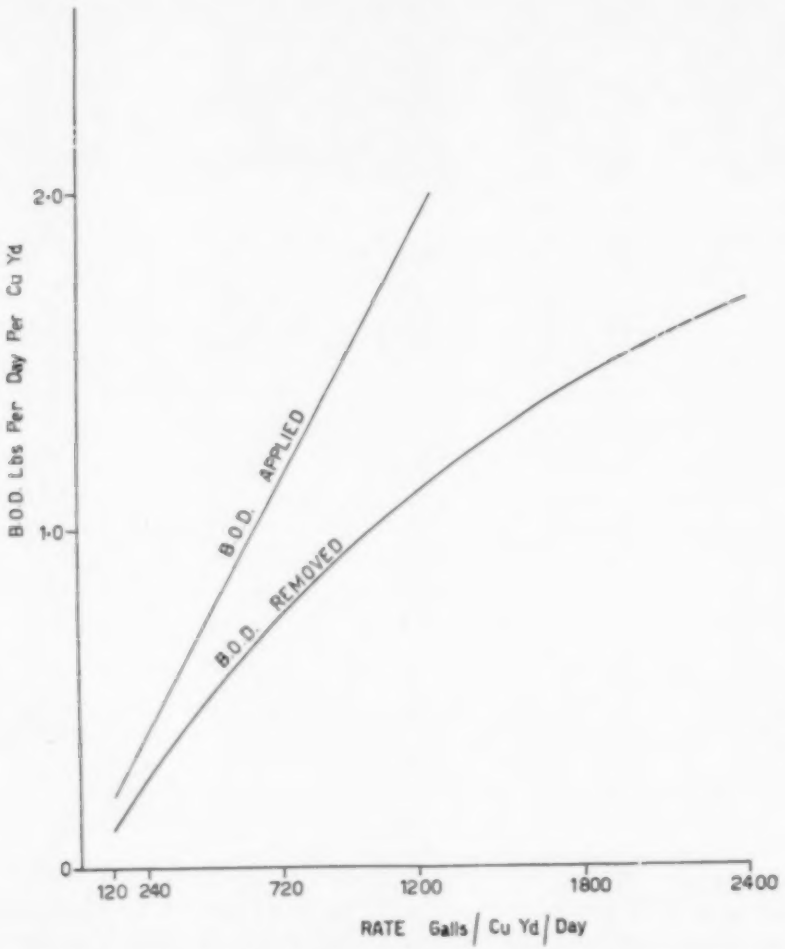


FIG. 4. Relationship between the BOD load applied and removed in the treatment of sewage at Minworth, Birmingham. (Reproduced by permission of the Controller of H. M. Stationery Office from the Annual Report of the Water Pollution Research Board (61).

and December (56). In January and February a thick growth of *Sepedonium* covered the surface and subsequently caused deterioration of the effluent.

Barraclough (4) experienced similar difficulties with clogging of a primary filter treating highly organic sewage at 800 gyd. With a weaker sewage Peach (46) obtained better results with double filtration than with alternating filtration or recirculation.

High rate filtration

High rate filtration of sewage is almost unknown in Great Britain. The rates of application with recirculation are within the conventional range.

An investigation of the filtration of Birmingham sewage at rates from 240 to 1200 gyd was carried out on a pilot plant by the Water Pollution Research Laboratory (61). Whereas ponding occurred at all rates with 3/4 to 1½ in. medium, rates of 720 gyd, equivalent to a loading of 0·92 lb B.O.D./yd³, could be applied with 1½ to 3 in. medium. Figure 4 shows the relationship between the B.O.D. load applied and removed in the pilot plant. These results are of practical interest. They suggest that high rate filtration might be used for the pre-treatment of industrial sewages before activated sludge treatment or biological filtration at conventional rates in order to deal with shock loads.

The high rate filtration of effluent from an activated sludge plant was investigated by Edmondson and Goodrich (9) at Sheffield in small and large scale experimental plants. Filters containing fine media dosed with activated sludge plant effluent at rates varying from 1000 to 1750 gyd gave a high degree of nitrification in pilot filters. With all rates the production of nitrate was almost constant at 0·24 lb oxidized nitrogen per yd³ per day. When the process was tried on a 30 ft diameter filter the efficiency of nitrification fell, due to the method of distribution. By applying the effluent through nozzles which gave a rain-like spray, a 300 per cent improvement in ammonia oxidation took place. The oxidized nitrogen rose from 0·06 lb/yd³ when intermittent dosage through jets was used to 0·16–0·21 lb when the effluent was sprayed.

Enclosed filtration

When the process of enclosed filtration was tried with the industrial sewage of Wolverhampton in a filter 20 ft diameter and 13 ft deep supplied with 20 ft³ of air per min the B.O.D. load in settled sewage with a B.O.D. of 325 ppm was removed at the rate of 0·52 lb/yd³. With 6 ft depth of medium rates of application of 1300 gyd were reached and 2·08 lb B.O.D. per yd³ representing 75 per cent of the total was removed (27).

At Glasgow part of the sewage has been treated since 1938 in an enclosed filter 52 ft diameter with 18 ft depth of medium. The enclosed filter gave a 70–90 per cent removal of B.O.D. at 350 gyd (26).

When milk wastes were treated in two pilot enclosed aerated filters, one with 13 ft and the other with 4½ ft of medium the results showed that if the feed to the

VOL. 5 1961

shallow filter was diluted to a B.O.D. of about 500 ppm and treated at 150 gall day-yd^3 the removal of B.O.D. was 0·31 1b yd^3-day. By applying the resultant effluent to the deep filter the B.O.D. was reduced to less than 20 ppm and the removal was 0·29 1b B.O.D. per yd^3 (30). The shallow filter was found to be capable of removing 0·5–0·8 1b B.O.D. per yd^3 when used for partial treatment of milk waste.

An interesting application of enclosed filters has recently been made at a milk waste treatment plant. When the alternating double filtration plant became overloaded two 24 ft diameter enclosed filters 9 ft deep were added to give pre-treatment to the screened raw waste. By applying a B.O.D. load of 4·37 1b yd^3–day and removing 2·45 1b the load has been reduced sufficiently to allow the original plant to continue in operation (15).

Sand filtration of effluents

The methods used for the further purification of effluents are lagooning, grassland treatment (31), micro-straining and sand filtration (10). At Luton effluent produced by activated sludge treatment followed by biological filtration and secondary settlement is further treated in rapid gravity sand filters at 240g/ft^2–hr. The filters are back washed twice every 24 hr. The B.O.D. is reduced from 13 to 7 ppm. Sand filtration is also used at Monsanto"s works (66) to reduce the suspended solid content of an activated sludge plant effluent having a B.O.D. of 6 ppm. The process cannot be relied upon to produce effluents of good bacterial quality (1) or to remove the eggs of pathogenic worms (50).

Biological investigations

British investigators have concluded that the grazing fauna in filters—such as fly larvae, the collembolan *Achorutes* and worms—were the chief agents controlling the organic film and were essential for the efficient operation of filters (33, 36, 54). This is illustrated by Fig. 5, which shows how with increasing temperature the increased grazing fauna brings about a reduction in the amount of film. Most American workers concluded that grazing fauna play a minor role (24, 25, 5,) possibly because at the higher rates applied in the U.S. they are of less importance.

Insect control

Lloyd and co-workers (36) found temperature to be important in determining the seasonal incidence of the different flies. Competition for food controlled the numbers of the different species. Industrial wastes could restrict the numbers of species able to breed in the filters and so in the absence of competition the remaining species were able to achieve high populations and thereby cause nuisance. The insecticide "Gammexane", which contains gamma benzene hexachloride, is used at Birmingham at a rate of 1·3 1b of the gamma isomer per acre of filter to control filter flies. The effects of insecticide treatment on the film and fauna balance were investigated. The film and fauna were assessed by removing samples of medium contained in perforated canisters. The weight of film was found by washing the medium

VOL.
5
1961

PLATE 1. Machine used for washing film from filter medium.

PLATE 2. Grid for separating and collecting fauna (left). Perforated canister and flag for indicating position of filter (right).

PLATE 3. Section of 9 acres of rectangular filters with rope hauled electrically-driven travelling distributors. The distributor in the foreground is discharging from two arms.

PLATE 4. Separate channels for collecting effluent from different sized media. Main effluent channel. Section of 9 acres of rectangular filters used for experimental purposes.

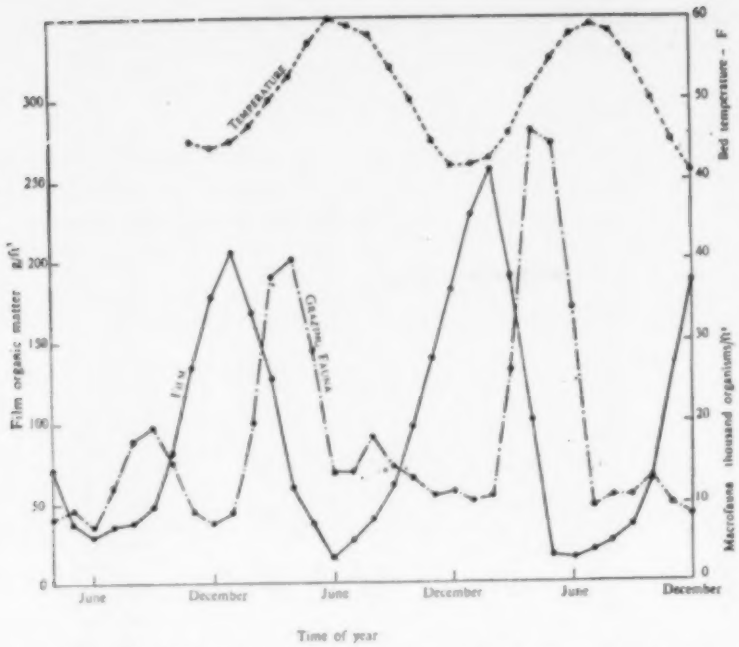


FIG. 5. Relationship between the seasonal accumulation of film and the incidence of grazing fauna in a filter treating domestic sewage at Langley, Birmingham. (Reproduced from a paper by Hawkes (21).

VOL. 5 1961

using a perforated drum (Plate 1). The fauna were removed and counted on a sectored illuminated counting chamber (Plate 2). Insecticide treatment, although effective in suppressing the fly population for some weeks following treatment, resulted in an increased accumulation of film because of the absence of grazers. This increased film accumulation was then available to successive generations of flies which unless suppressed by still further treatment were able to attain higher populations than in untreated filters. In the absence of insecticide treatment the film had been reduced by the active grazing fauna (16). In some filters several applications of insecticide produced excessive accumulation of film which eventually caused choking of the filter. In other filters on which sewage was sprayed continuously insecticide treatment increased the worm population and the springtail *Achorutes subviaticus*. These organisms grazed the film more effectively than fluctuating fly populations and the filter became cleaner and more efficient (17). It was concluded that the best way to control the fly population was to control their food supply by limiting the accumulation of film (21).

A comparison between two similar filters which contained different amounts of film showed that the one with the smaller amount was the more efficient (19). Only a very thin slime is needed for efficient purification. This has been demonstrated by growing a biological film in a rotating tube (65) and determining the rate of oxidation of glucose at different stages of growth. The maximum rate of oxidation

was reached with 15 g of film with a thickness of 0·25 mm and it did not increase even when 100 g had been grown.

Operational investigations

In 1946–49 plans were prepared to enable 9 acres of beds to operate by alternating double filtration (6) using distributors which travel forwards and backwards (Plate 3). Where such machines are used the interval between successive doses is

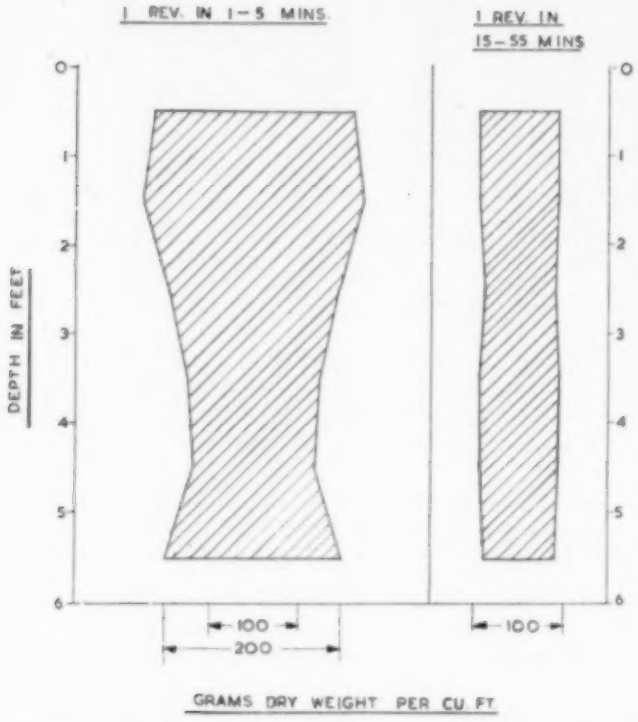


Fig. 6. Difference between the film accumulated at different depths in filters with (left) a self-propelled distributor and (right) a slowly revolving mechanically driven distributor. (From data in a paper by Tomlinson and Hall (55).

relatively long but the four-arm rotary distributors used for the experiments on alternating filtration made one revolution in 1–2 min. To test the effect of varying the time interval between successive doses one pair of 120 ft diameter filters was fitted with a geared drive, which enabled the arms to make one revolution in either 8, 15, 30, 42 or 55 min. Over an experimental period of 5 years the conditions were varied so as to determine the effect of the dosing cycle (55). The results were compared with two filters dosed from a jet-impelled distributor making one revolution in from 1 to 5 min.

The conclusion reached was that a distributor making one revolution in 15–30 min gave the best results and confirmed the observations by Lumb and Barnes (39) that by slowing down the rate of revolution of a distributor less surface growth was produced. To explain this unexpected conclusion shafts containing baskets of medium were sunk into the filters and the medium subsequently examined for

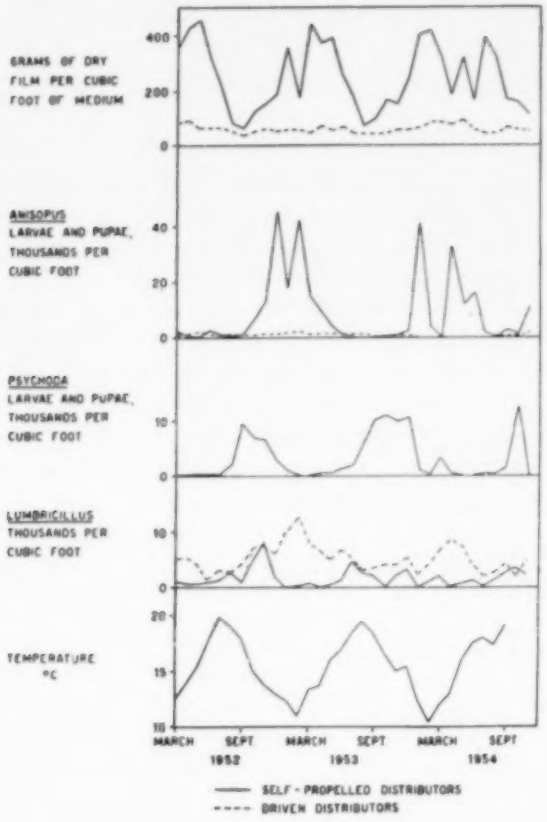


FIG. 7. Difference between the seasonal accumulation of film, and the numbers of grazing fauna in the upper 2 ft of (a) a filter with a self-propelled distributor and (b) a filter with a slowly revolving mechanically driven distributor. (Reproduced from a paper by Hawkes (18).

the amount and type of growth. The results summarized in Fig. 6 show that slowly revolving distributors reduced the amount of growth and that this was more evenly distributed throughout the depth of the filter. The reduced amount of growth oxidized the organic impurity and ammoniacal nitrogen with greater efficiency than the control filter growth.

The film and fauna fluctuations were also assessed (18). The results given in Fig. 7 show that the amounts of film in the surface layers of the filter dosed from

VOL. 5 1961

a slowly revolving distributor remained at a uniformly low level whereas in the control filter the amount of growth showed wide seasonal variations. Another marked effect was the replacement of the population by one dominated by the worm *Lumbricillus lineatus*.

Recent work (64) has shown that the time of contact is longer when intermittent dosing is used than with continuous dosing. The reason suggested is that with continuous dosing the salt used in the test is washed out of the filter at a steady rate. With intermittent dosing the first flush of effluent brings out part of the salt and several more flushes are needed to wash out the whole of the salt.

Experiments at rates of flow varying during each day

Over a considerable period the rate of flow was varied throughout the day so as to simulate normal variations. From 6 a.m. to 2 p.m. the rate was 90 gyd; from 2 p.m. to 10 p.m. 270 gyd and from 10 p.m. to 6 a.m. 180 gyd, giving an overall rate of 180 gyd. Typical results given in Table 4 show that the BOD of the filter effluent from the primary bed (which receives the sewage at double the rates given above) deteriorates more in consequence of an increase in strength of the feed liquor than as a result of an increase in the rate of application.

During the past two years experiments on four 120 ft diameter filters have been concerned with comparisons between double filtration with and without alternation. Two filters were fitted with variable speed gear driven distributors capable of being rotated at different speeds between one revolution in 1 and 60 min. They were matured carefully by running them for a year as single filters. The remaining two filters were operated by alternating filtration. The two different methods of opera-

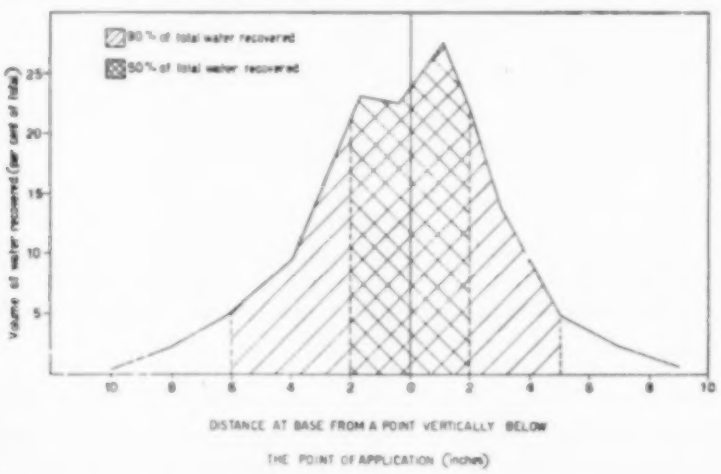


Fig. 8. Distribution of water at the base of a 6 ft deep filter filled with clean medium, after water had been applied along a line at the surface. (Reproduced by permission of the Controller of H. M. Stationery Office from the Annual Report of the Water Pollution Research Board (62).

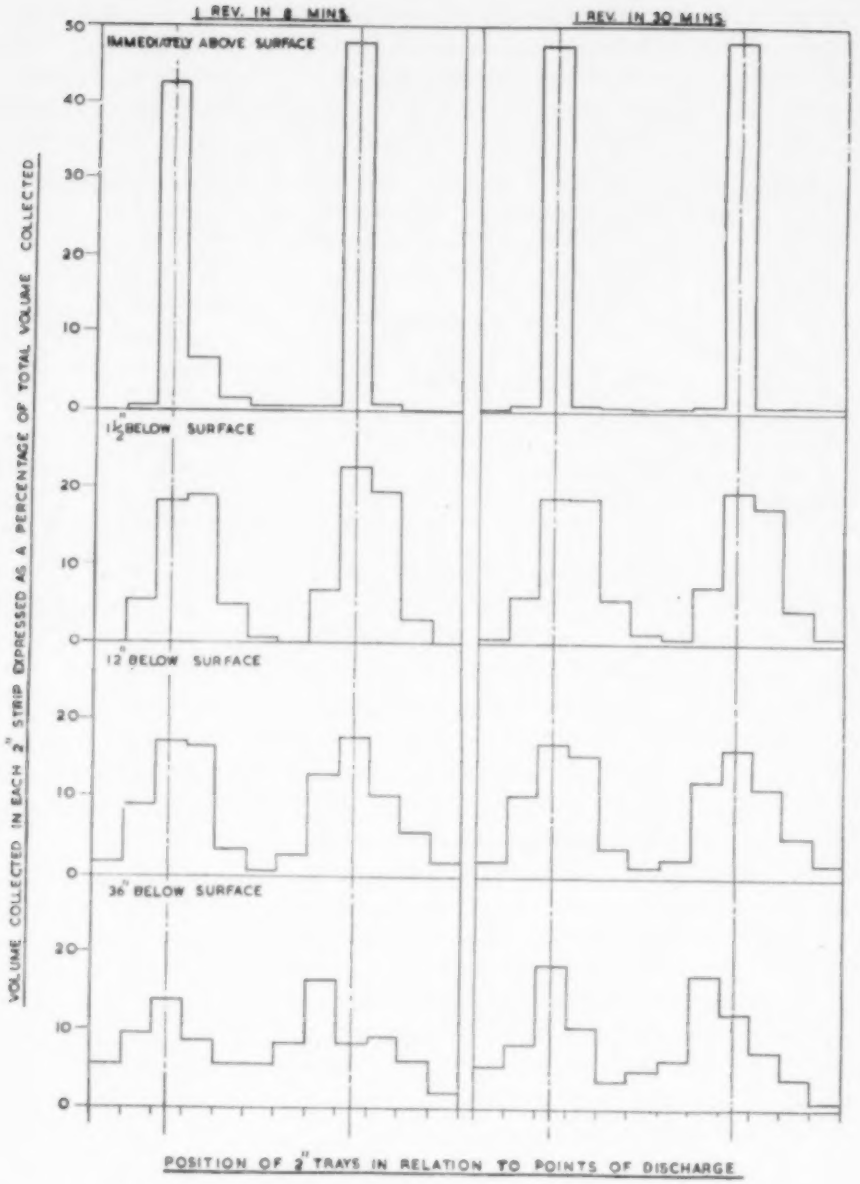

1 REV. IN 8 MINS.
1 REV. IN 30 MINS.
IMMEDIATELY ABOVE SURFACE
1½" BELOW SURFACE
12" BELOW SURFACE
36" BELOW SURFACE
VOLUME COLLECTED IN EACH 2" STRIP EXPRESSED AS A PERCENTAGE OF TOTAL VOLUME COLLECTED
POSITION OF 2" TRAYS IN RELATION TO POINTS OF DISCHARGE

FIG. 9. Distribution of sewage at different depths after being applied from adjacent jets 12 in. apart to the surface of a mature filter. The results are shown when the distributor rotated once in 8 min and once in 30 min.

VOL. 5 1961

tion gave effluents of approximately the same composition but with a slightly lower B.O.D. and slightly higher nitrate content in the effluents from the double filters without alternation. It is suggested that double filtration would be successful if growth on the primary filter is minimized by controlling the frequency of dosing.

Table 4. Treatment of settled sewage at Minworth, Birmingham, by alternating double filtration applying three rates of treatment each day. Results for year ending November 1947 (60)

Time	Rate of treatment each day (gall/yd³–day)	B.O.D. (ppm)		
		Feed liquor	Settled effluent	
			Primary filter	Secondary filter
6 a.m.–2 p.m.	90	158	14	11
2 p.m.–10 p.m.	270	144	21	11
10 p.m.–6 a.m.	180	174	32	14

The Water Pollution Research Laboratory found that 90 per cent of water applied at 375 gyd on a line at the surface of a clean filter 6 ft deep containing $\frac{3}{4}$–$1\frac{1}{2}$ in. clinker emerged in a band about 11 in. wide, and 50 per cent within 2 in. of either side of the point of application, as illustrated in Fig. 8 (62).

The distribution of sewage below the surface of a mature filter was determined by collecting the effluent in 12 separate adjacent compartments each 2 in. wide so placed as to lie parallel, i.e. tangentially, to the path of the distributor jets. Some of the results obtained at three depths under two adjacent jets 12 in. apart at different rates of revolution of the distributor are given in Fig. 9.

There was an immediate limited lateral spread on impinging on the surface as shown by the distribution at $1\frac{1}{2}$ in. depth. The displacement of the collected effluent to the right was due to the centrifugal force of the rotating distributor. Below the surface the lateral spread was more gradual but even at 3 ft distribution was still uneven. There was indication that with the more slowly revolving distributor the lateral spread was less than with the quickly revolving distributor. In this filter, which is provided with side sampling pipes, the times of contact determined by the salt method when the dosage was 375 gyd were 9·5, 15·3 and 20·7 min at depths of 2 ft, 4 ft and 6 ft respectively.

OBSERVATIONS AND EXPERIMENTS ON LARGE SCALE PLANT OPERATING ON ALTERNATING DOUBLE FILTRATION

In order to provide information needed for the adaptation to alternating filtration of a further 36 acres of filters observations were made on the biological conditions in the 9 acres of filters already operating by this method and of their efficiency. The dosage rate was limited to 130–160 gyd during this period.

Seasonal fluctuations in film accumulation

The amounts of film and fauna in the gacres were compared with those on the experimental 120 ft diameter alternating double filters receiving sewage at a higher rate from distributors revolving once in 30 min. The results of comparisons given in Fig. 10 show that during the first winter 1954–5, which was relatively mild, no excessive film accumulation took place in the large rectangular filters although the amount was greater than in the experimental filters. The next winter was more severe and early in the winter the large filters were out of operation for a period. These conditions depleted the fauna and resulted in an excessive accumulation of film (19).

Figure 10 shows the corresponding fluctuations in efficiency measured by the percentage B.O.D. removal and nitrification. During 1954–5 when little film accumulated, the B.O.D. removal in the primary stage in the large filters was hardly affected. In the second winter there was a marked drop in efficiency, corresponding to heavy

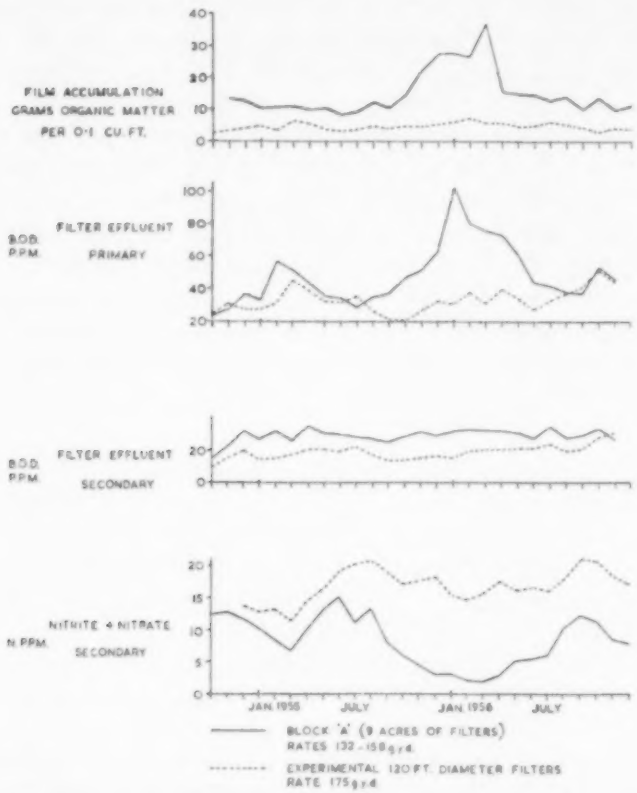


FIG. 10. Comparison between the accumulation of film and the efficiency of 120 ft. diameter experimental filters operating on alternating double filtration at 175 gal/day-yd^2 applied from distributors revolving once in 30 min with the results obtained on 9 acres of filters operating at 132–158 gal/day-yd^2, dosed from distributors travelling forward and backward once in 6½ min.

VOL. 5 1961

film accumulation. When the primary stage was effective the secondary stage dealt with a very weak liquor and it is presumed that the film passed into the endogenous state and became reduced in amount. However, when the primary stage became less effective the secondary filter received more organic impurity. Although the effluent from this filter did not greatly deteriorate, because the filter operated at greater efficiency, the amount of organic matter with which the secondary filter was supplied as primary effluent enabled the film to grow, instead of to decrease, in amount. For this reason, as the filters became alternately primary and secondary, the increments of growth obtained in the secondary stage finally resulted in such an accumulation on both filters as to cause ponding. More recent experience has confirmed the validity of the view that alternating filtration is successful only if the secondary stage is used for film control (42).

In the 9 acres of alternating filters difficulties have been found in obtaining satisfactory results because of excessive film growth. In the 120 ft diameter filters this build up of film is avoided by periodicity of dosing and the sewage has been well purified. The importance of preventing this accumulation of film led to investigations on the effect of the method of applying the sewage and of the size of medium on the development of film, the macrofauna and the efficiency of treatment. One part of the filtration area was fitted with splash plates under the jets to cause the sewage to spread out and wet all the filter surface. It appeared that in summer and in mild winters there was a greater grazing fauna and less film under splash plates than under ordinary jets and the effluent from the area with splash plates was superior in B.O.D. and oxidized nitrogen. Under severe conditions of low temperature and whenever the fauna was depleted less, film was present in the area without splash plates and the effluent B.O.D. was then less than from the area with splash plates. Both effluents contained little oxidized nitrogen compounds.

Tests with different kinds of jets

Six different types of jet were used on one distributor and the film growth and fauna population assessed once a month over a period of 14 months (20). The jets discharged equivalent amounts of sewage and were modified as follows.

(A) *Open fish tail.* The jet from the original 7/8 in diameter nozzle was discharged over a triangular tray to give a sheet of liquid approximately 24 in. wide on the surface of the bed.

(B) *Closed fish tail.* The sewage was discharged through a rectangular orifice 9 in. long and 1/4 in. wide to produce a sheet of liquid approximately 24 in. wide but with a somewhat stronger impinging force than in (A).

(C) *Splash plate.* The jet discharged on to a circular disc 2½ in. diameter which produced a circular sheet approximately 24 in. diameter.

(D) *The unmodified nozzle.* This gave a cylindrical jet of liquid from a 7/8 in diameter hole.

(E) *Twin jets.* The jet in (D) was divided so as to discharge through two jets 12 in. apart.

(F) *"Quad" jets.* The jet in (D) was divided into four jets 6 in. apart.

The results in Figs. 11 a, b and c show the relationship between the amount of film and fauna both on the surface and 6 in. immediately below the surface. Surface growth was related to the evenness of distribution with evidence that the impinging force of the single jet suppressed the surface growth. Below the surface

Fig. 11a.

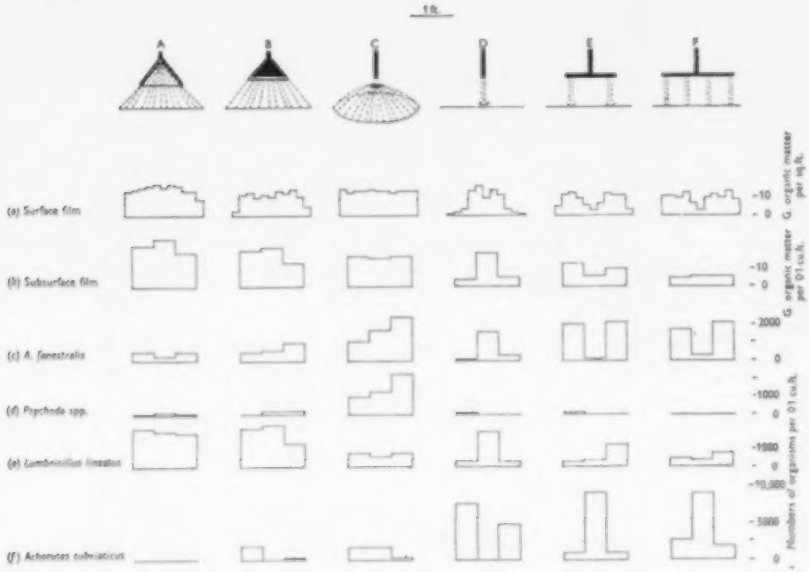


Fig. 11b.

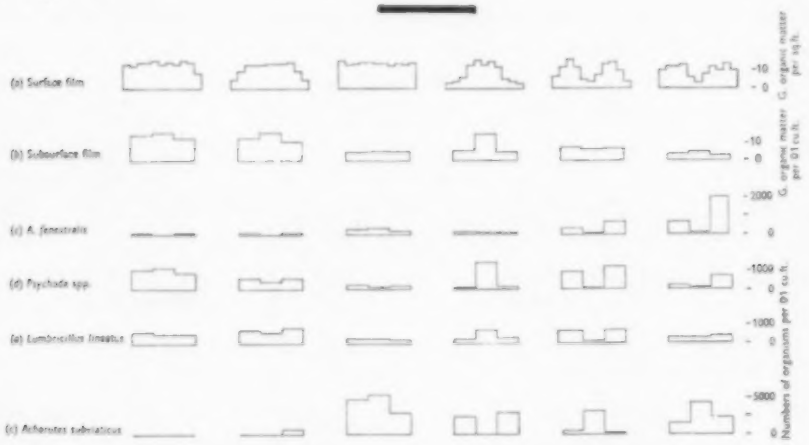

(a) Surface film
(b) Subsurface film
(c) A. fenestralis
(d) Psychoda spp.
(e) Lumbricillus lineatus
(f) Achorutes subviaticus
G. organic matter per sq. ft.
G. organic matter per 01 cu. ft.
Numbers of organisms per 01 cu. ft.

FIG. 11. Effect of different methods of distribution of sewage on the amount of surface and sub-surface film and the numbers of grazing organisms in the sub-surface layer of medium (20), (a) Mild winter conditions while the filter was operated by alternating double filtration, (b) Severe winter conditions, while the filter was operated by single filtration, when the loading was higher than in (a).

VOL. 5 1961

FIG. 11c.

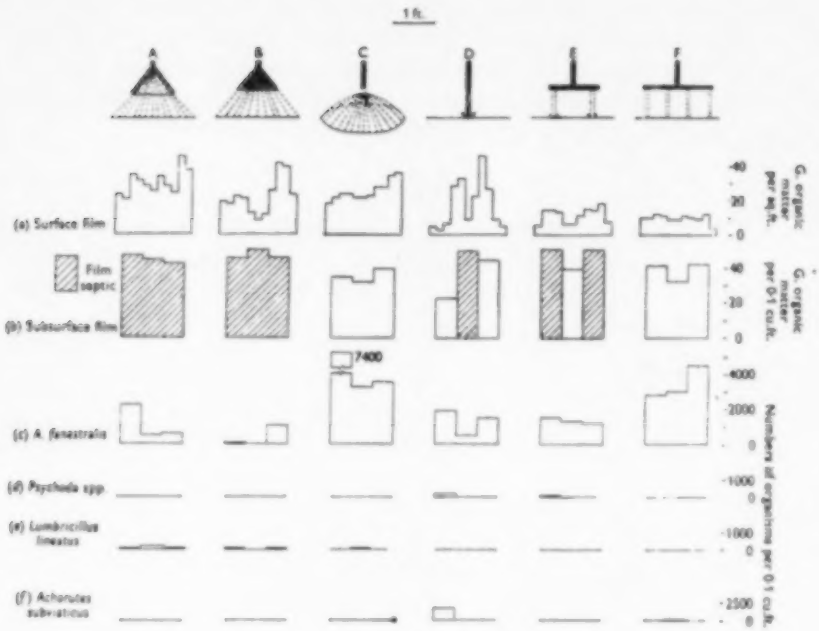


FIG. 11 *(continued from previous page)*
(c). Summer conditions while the filter was operated by alternating double filtration.

the flush of sewage had a greater suppressing effect on the fauna than on the amount of film. The collembolan *Achorutes* was rare under the stronger flushes from (A) and (B) and most common between the jet lines of (D), (E) and (F). Fly larvae were most common in areas of reduced flow produced either by the splitting of jets or by splash plates. In (E) and (F) they were under the jet and thus occupied a different ecological niche to *Achorutes* which as previously stated were confined to the inter-jet zones. Thus the two populations, fly larvae and *Achorutes*, were able to co-exist. Under the splash plate *Achorutes* and fly larvae were in direct competition for food and this resulted in each becoming successively dominant according to the season. Since the maintenance of a mixed grazing fauna is desirable it was considered that distribution from the jet split into four sub-jets gave the best surface and sub-surface conditions.

The indirect effect of the fauna on the sub-surface film is evident. In the absence of interspecific competition worms were abundant but they were not able to control film growth under the strong flushing action from the fish tail jets. Consequently more film accumulated under these jets as a result of the decreased grazing activity. Under severe winter conditions this led to ponding along the strips of filter served by these jets.

Apart from their obvious practical implications these observations demonstrate that the flushing action of sewage applied at the fairly high instantaneous rates which apply in some cases in the U.K. does not of itself result in the mechanical scouring of the film. The importance of grazing fauna in controlling film accumulation under conditions common in Great Britain is now well established and confirms the view expressed by Reynoldson (47).

Effect of size of media on film growth and purification

One-eighth of the 9 acres of filters was divided into four separate sections each with its own effluent collecting channel (Plate 4). The sections contained (1) the original 1¼ in. cracked granite, (2) 1½ in. crushed gravel, (3) 1½ in. round gravel, and (4) 2½ in. round gravel. Eight slotted steel shafts containing perforated canisters filled with medium were sunk into the four sections of the beds so that the rate of growth of film and its biological composition could be determined under the distributor jets. The filter was operated for about a year as a single filter (22) and for two years by alternating filtration (23). The results obtained are summarized in Table 5.

TABLE 5. AVERAGE RESULTS OF ANALYSIS AND CHEMICAL AND BIOLOGICAL COMPOSITION OF FILM FROM FILTERS CONTAINING DIFFERENT GRADES OF MEDIUM. THE FILTERS WERE OPERATED FOR A PERIOD AS SINGLE FILTERS AND SUBSEQUENTLY AS ALTERNATING DOUBLE FILTERS

	Filter medium			
	1¼ in. cracked gravel	1½ in. crushed gravel	1½ in. round gravel	2½ in. round gravel
Single Filtration 1945–55				
Organic film, g dry matter per 0·1 ft²	12·4	9·8	8·4	6·6
B.O.D. settled effluent (ppm)	20·4	21·9	34·8	37·5
Nitrite + Nitrate N (ppm)	7·4	5·1	4·1	2·9
Numbers of *Anisopus, Psychoda* and *Lumbricillus* per 0·1 ft²	802	360	394	222
Alternating Double Filtration Winter Conditions 1955–56				
Organic film g dry matter per 0·1 ft²	24·0	17·9	11·1	13·0
B.O.D. settled primary effluent (ppm)	72·4	57·6	55·8	56·4
B.O.D. settled secondary effluent (ppm)	34·2	30·5	27·9	27·7
Numbers of *Anisopus, Psychoda* and *Lumbricillus* per 0·1 ft²	86	108	233	213
Alternating Double Filtration Summer Conditions 1957				
Organic film g dry matter per 0·1 ft²	13·2	12·6	10·9	8·3
B.O.D. settled primary effluent (ppm)	36·0	37·2	39·3	39·7
B.O.D. settled secondary effluent (ppm)	30·4	31·4	32·3	29·2
Numbers of *Anisopus, Psychoda* and *Lumbricillus* per 0·1 ft²	1565	1343	1588	545

VOL. 5 1961

With single filtration the best effluents were obtained with the original medium (1½ in. cracked gravel), which supported the richest fauna, and the worst with 2½ in. round gravel. However, the original medium ponded badly during winter and the heavier volumetric loading applied with alternating double filtration flooded it completely resulting in a marked deterioration in the quality of the effluent. The larger medium allowed surface growth to develop without ponding and during the

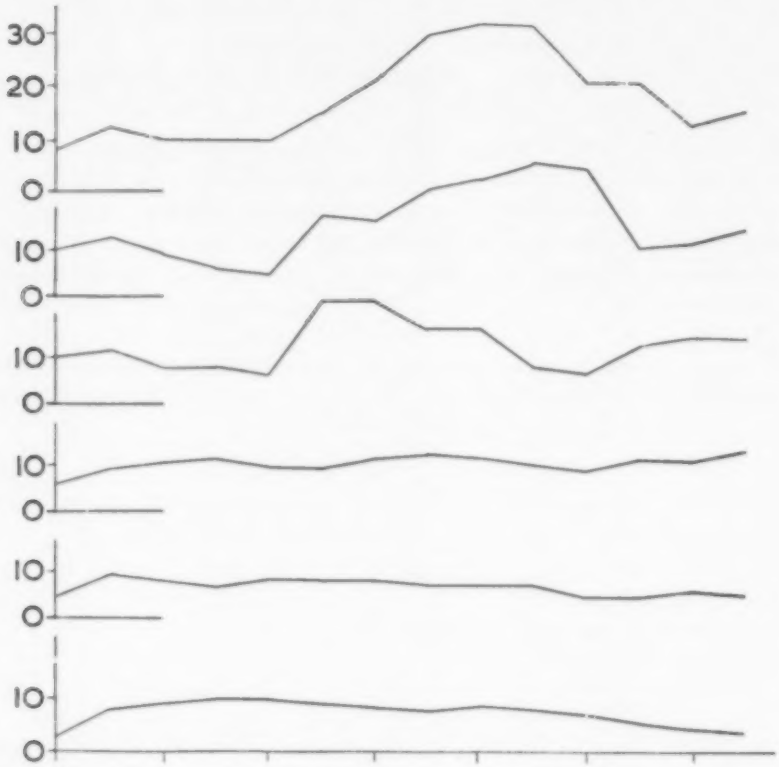


Fig. 12. Seasonal fluctuation in film accumulation at different depths in a filter.

winter period gave primary effluents superior to those from the smaller media. The amount of growth found on the different grades of media at different seasons varied according to the depth, as shown in Fig. 12.

It will be seen that the winter accumulation of film took place mostly in the upper layers.

DISCUSSION AND SUMMARY

The development of biological filtration in Great Britain during the past 25 years has advanced along three lines: (1) practical experiments on modified systems of filtration, (2) biological studies in connection with particular methods of im-

proving filtration and of a general ecological nature, (3) fundamental investigation of the dynamics of oxygen transfer during filtration.

The volumetric loading of filters in America is generally higher than in Britain because of differences in geographical conditions, water consumption and strength of sewage and the degree of treatment required. These differences may account for some of the contrary views expressed on the relative importance of the different factors which affect the efficiency of biological filtration. Some American workers appear to regard the discharge of film from filters as due largely to the scouring action of the sewage. British workers stress the importance of larger forms of life such as worms and fly larvae in converting organic film into removable humus. The part played by bacteria and microorganisms such as Rotifers and Nematode worms in controlling the growth of film has yet to be determined.

In one respect practice had led theory in that two filtration processes, recirculation and alternating double filtration, both involve the principle of limiting the film by nutritional control. In recirculation the diluted feed results in less growth of film. In alternating double filtration the starvation of the film in the endogenous secondary stage results in a reduction in the film previously developed in the primary stage. These processes were successfully practised before the fundamental principles were adequately understood. When such processes fail, successful action is more likely to be taken if the underlying principles of the process are appreciated.

Although the importance of the macrofauna in removing film is now acknowledged, the more recent tendency in filter operation in Great Britain is to prevent the accumulation of film in the first place. Under winter conditions the grazing fauna cannot always be relied upon to achieve this objective. Nutritional control of the film by recirculation, alternating double filtration and periodicity of dosing then appear to offer prospects of limiting film growth and thereby to ensure efficiency of treatment and control of fly nuisance.

VOL. 5 1961

Acknowledgements—This paper is presented by permission of Mr. M. R. Vincent Daviss, Engineer to the Birmingham Tame and Rea District Drainage Board. The authors would like to thank the Engineer's drawing office and laboratory staff for their assistance in the preparation of this paper.

REFERENCES

1. Allen L. A., Tomlinson T. G. and Norton I. L. *J. Inst. Sew. Purif.* 115 (1944).
2. Applewhite A. *J. Inst. Sew. Purif.* 116 (1935).
3. Ardern E. and Lockett W. T. *J. Soc. Chem. Ind.* **33**, No. 10 (1914).
4. Barraclough D. H. *J. Inst. Sew. Purif.* 361 (1954).
5. Cooke W. B. and Hirsch A. *Sew. Industr. Wastes* **30**, 138 (1958).
6. Daviss M. R. V. *Sew. Industr. Wastes* **23**, 437 (1951).
7. Dickinson D. *J. Inst. Sew. Purif.* Pt. 1, 54 (1949).
8. Edmondson J. H. and Goodrich S. R. *J. Inst. Sew. Purif.* Pt. 2, 35 (1947).
9. Edmondson J. H. and Goodrich S. R. *J. Inst. Sew. Purif.* 37 (1943).
10. Evans S. C. and Roberts F. W. *J. Inst. Sew. Purif.* 225 (1955).

11. FOWLER G. J. and HOLTON A. L. *J. Soc. Chem. Ind. Lond.* **30**, 180 (1911).
12. GOLDTHORPE H. H. and NIXON J. *J. Inst. Sew. Purif.* 101 (1942).
13. GREENE G. *J. Inst. Sew. Purif.* 116 (1957).
14. HALL J. T. *Proc. Assoc. Sew. Wks. Man.* 143 (1930).
15. HARTLEY (Stoke-on-Trent) Ltd., Private communication (1959).
16. HAWKES H. A. *Ann. Appl. Biol.* **39**, 181 (1952).
17. HAWKES H. A. *Ann. Appl. Biol.* **43**, 122 (1955).
18. HAWKES H. A. *J. Inst. Sew. Purif.* Pt. 1, 48 (1955).
19. HAWKES H. A. *J. Inst. Sew. Purif.* Pt. 2, 88 (1957).
20. HAWKES H. A. *Ann. Appl. Biol.* **47**, 339 (1959).
21. HAWKES H. A. *Symposium on the Treatment of Waste Waters*, Univ. of Durham, Edited by P. C. G. Isaac (1959).
22. HAWKES H. A. and JENKINS S. H. *J. Inst. Sew. Purif.* Pt. 4, 352 (1955).
23. HAWKES H. A. and JENKINS S. H. *J. Inst. Sew. Purif.* Pt. 2, 221 (1958).
24. HEUKELEKIAN H. *Sew. Wks. J.* **17**, 23 (1945).
25. HOLTJE R. H. *Sew. Wks. J.* **15**, 14 (1943).
26. HUNTER A. and COCKBURN T. *J. Inst. Sew. Purif.* 12 (1944).
27. HURLEY J. and LOVETT M. *J. Inst. Sew. Purif.* Pt. 2, 11 (1938).
28. JENKINS S. H. *J. Inst. Sew. Purif.* Pt. 1, 206 (1937).
29. JENKINS S. H. *Biochem. J.* **25**, 147 (1931).
30. JENKINS S. H. Private report to one of the authors (SHJ).
31. JENKINS S. H. *J. Inst. Pub. Health Eng.* (1960) 29, LIX.
32. JENKINS S. H. *J. Inst. Sew. Purif.* 224 (1954).
33. JOHNSON J. W. H. *J. Econ. Bio.* **9**, 105, 127 (1941).
34. LAMBDEN A. E. *J. Inst. Sew. Purif.* 174 (1959).
35. LECLERC E. *Symposium on the Treatment of Waste Waters*. Univ. of Durham, Edited by P.C.G. Isaac (1959).
36. LLOYD L. *J. Inst. Sew. Purif.* Pt. 2, 119 (1945).
37. LUMB C. *J. Inst. Sew. Purif.* Pt. 4, 383 (1956).
38. LUMB C. *Symposium on the Treatment of Waste Waters*. Univ. of Durham, Edited by P.C.G. Isaac (1959).
39. LUMB C. and BARNES J. P. *J. Inst. Sew. Purif.* Pt. 1, 83 (1948).
40. LUMB C. and EASTWOOD P. K. *J. Inst. Sew. Purif.* Pt. 4, 380 (1958).
41. MILLS E. V. *J. Inst. Sew. Purif.* Pt. 2, 94 (1945).
42. MILLS E. V. *J. Inst. Sew. Purif.* Pt. 2, 35 (1945).
43. MUERS M. M. Private communication; *see* JENKINS S. H. *The Treatment of Trade Waste Waters and the Prevention of River Pollution*, edited by P. C. G. Isaac, 115 (1957).
44. OLIVER F. *J. Inst. Sew. Purif.* Pt. 3, 263 (1949).
45. O'SHAUGHNESSY F. R. *Proc. Assoc. Sew. Wks. Man.* Pt. 1, 74 (1931).
46. PEACH J. D. *J. Inst. Sew. Purif.* 111 (1957).
47. REYNOLDSON T. B. *J. Inst. Sew. Purif.* Pt. 1, 109 (1941).
48. REYNOLDSON T. B. *J. Inst. Sew. Purif.* Pt. 1, 116 (1942).
49. ROBERTS S. J. *J. Inst. Sew. Purif.* Pt. 2, 76 (1946).
50. SILVERMAN R. H. and GRIFFITHS R. B. *Ann. trop. Parasit.* **49**, 4, 436 (1955).
51. STOKER E. C. *J. Inst. Sew. Purif.* Pt. 1, 118 (1937).
52. TIDSWELL M. A. *J. Inst. Sew. Purif.* (1959) *In press.*
53. TOMLINSON T. G. *J. Inst. Sew. Purif.* Pt. 1, 39 (1941).
54. TOMLINSON T. G. *J. Inst. Sew. Purif.* Pt. 1, 168 (1946).
55. TOMLINSON T. G. and HALL H. *J. Inst. Sew. Purif.* 40 (1955).
56. TOMLINSON T. G. and HALL H. *J. Inst. Sew. Purif.* 110 (1951).
57. VELZ C. J. *Sewage Wks. J.* **20**, 607 (1948).
58. VOKES F. C. *J. Inst. Sew. Purif.* 220 (1954).

59. Water Pollution Research Technical Paper No. 8 H. M. Stationery Office, London (1941).
60. Water Pollution Research Report of the Director, 15 (1947).
61. Water Pollution Research Report of the Director, 13 (1951).
62. Water Pollution Research Report of the Director, 14 (1953).
63. Water Pollution Research Report of the Director, 60 (1955).
64. Water Pollution Research Report of the Director, 41 (1956).
65. Water Pollution Research Report of the Director, 63 (1957).
66. Wilson I. S. *Symposium on the Treatment of Waste Waters,* Univ. of Durham, Edited by P. C. G. Isaac (1959).

VOL.
5
1961